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Corte Madera Creek, Marin County, California, Modified Unit 4 Sedimentation Study

Numerical Model Investigation

Ronald R. Copeland

August 2000

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Preface

The numerical model investigation of Corte Madera Creek, reported herein, was conducted at the U.S. Army Engineer Research and Development Center (ERDC) at the request of the U.S. Army Engineer District, San Francisco (SPN).

This investigation was conducted during the period September 1998 to January 2000 in the ERDC Coastal and Hydraulics Laboratory (CHL) under the direction of Dr. James R. Houston, former Director of CHL, Dr. Phil G. Combs, former Chief of the Rivers and Structures Division, and Dr. Yen-Hsi Chu, Chief of the River Sedimentation Engineering Branch. The project engineer for this study and author of this report was Dr. Ronald R. Copeland. Mrs. Peggy Hoffman and Mrs. Dinah McComas provided technical assistance.

Mr. William Firth served as the hydraulic project engineer in SPN, providing valuable contributions and review during the course of the study.

At the time of publication of this report, Dr. James R. Houston was Director of ERDC, and COL James S. Weller, EN, was Commander.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
Acre-feet	4,046.873	Cubic meters
Cubic feet	0.02831685	Cubic meters
Cubic yards	0.7645549	Cubic meters
Feet	0.3048	Meter
Inches	2.54	Centimeters
Miles(U.S. statute)	1.609357	Kilometers
Square miles	2.589998	Square kilometers

1 Introduction

Background

Corte Madera Creek drains an area of approximately 28 square miles[1] in Marin County, California. The creek discharges into San Francisco Bay about 9 miles north of the Golden Gate. Between 1914 and 1960 there were 12 damaging floods from Corte Madera Creek. It was decided that a flood control project was needed because of the frequent flooding. Flood Control Zone Nine, an entity of Marin County, was created to be the local sponsor for a federal flood control project on Corte Madera Creek.

The project was designed to contain the Standard Project Flood, which is about 7,500 cfs at the Ross gauge and 9,700 cfs at San Francisco Bay. In 1972, the U.S. Army Corps of Engineers completed the first three units of the Corte Madera Creek flood control project. The completed portion of the project extended from San Francisco Bay through the cities of Corte Madera, Larkspur, Kentfield, and Ross, a distance of about 4.5 miles (Figure 1). The first 2.2 miles of the project is an earthen channel, dredged to el -12.0[2] with a bottom width of 80 ft and side slopes of 1V: 6H. The next 0.7 mile of earthen channel has a bottom slope of 0.0007, a 30-ft bottom width, and 1V: 6H side slopes. The next mile of the project consists of a 33-ft-wide concrete channel with a stilling basin at the downstream end. The first 1,000 ft of this channel has a mild slope of 0.0007. The remainder of the concrete channel has a steep slope of 0.0038 and is designed for supercritical flow.

The final unit of the flood control project, Unit 4, was to be an additional 3,000 ft of concrete channel. Construction of Unit 4 was delayed because of litigation, environmental concerns, and strong public opposition. An alternative plan for Unit 4 was then developed. This plan was designed to prevent damages from the Standard Project Flood and to preserve the ecological character of the creek. After extensive coordination the plan received public support. However, in 1980, before the plan was approved, the Marin County Board of Supervisors

[1] A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page viii.
[2] All elevations (el) cited herein are in feet referred to National Geodetic Vertical Datum (NGVD)

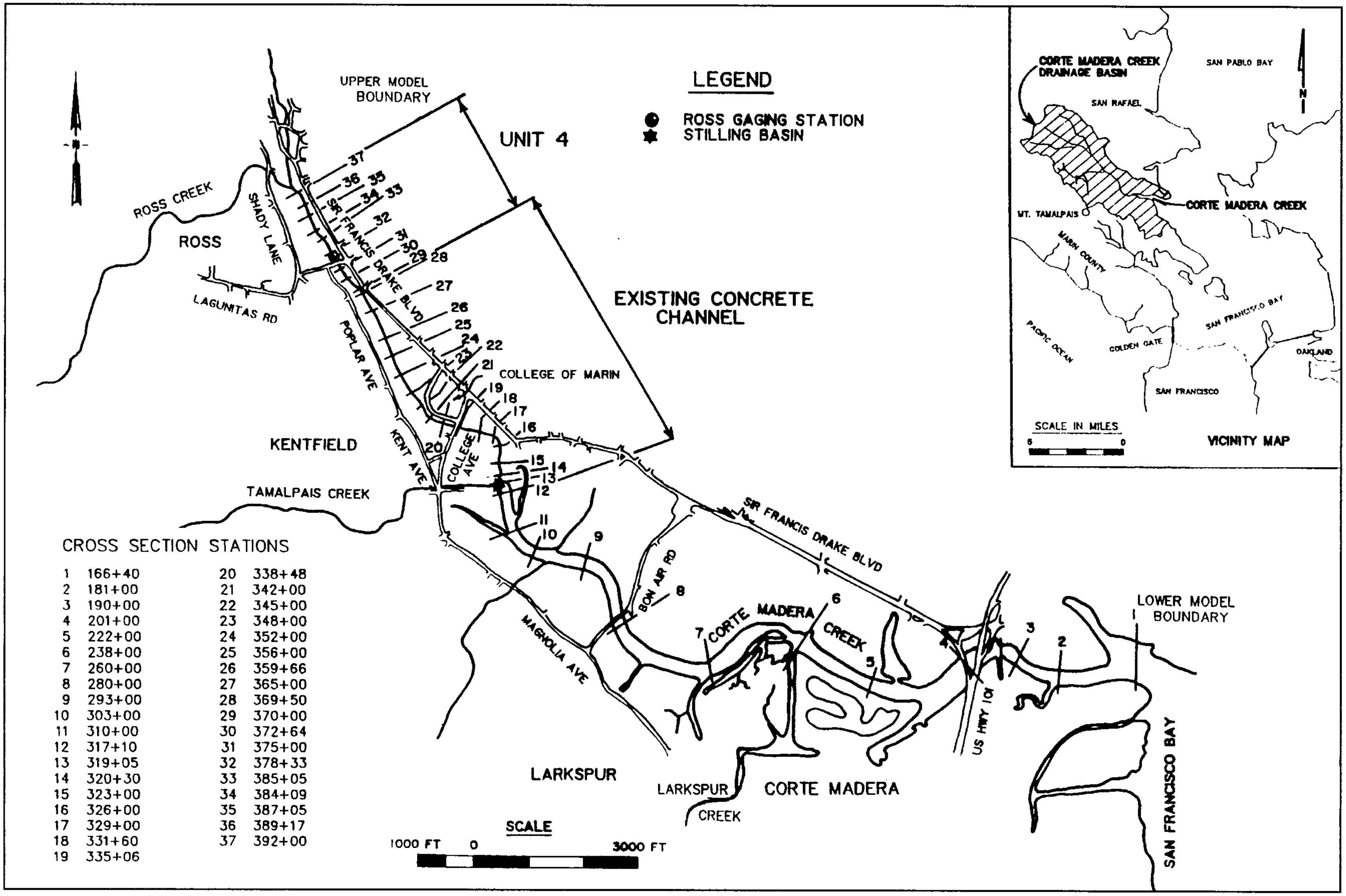

Figure 1. Cross-section locations

withdrew local sponsorship because of budget concerns related to California voter approval of Proposition 13. As a result, the Corps suspended engineering and design work.

Currently, flooding from Corte Madera Creek is primarily caused by the insufficient channel capacity in the Unit 4 reach. Flows greater than approximately 3,000 cfs overtop the right descending bank from upstream of the Lagunitas Bridge to the upstream end of the existing concrete channel. These flood flows proceed down Poplar and Kent Avenues inundating areas adjacent to the existing channel improvements in Ross, Kentfield, and the College of Marin. In the downstream reaches of the concrete channel, capacity is less than the original design discharge because of unanticipated increases in channel roughness and sediment deposition in the lower reaches of the channel. Increases in roughness are caused by the presence of barnacles and tube worms on the channel walls, by the accumulation of sands and gravels on the channel bed, and by abrasion of the concrete channel invert.

The largest recorded flood on Corte Madera Creek occurred in January 1982. This flood had an estimated peak of 7,200 cfs at the Ross gauge and a recurrence frequency greater than 100 years. Another flood occurred in March 1983, with an estimated peak of 3,480 cfs and a recurrence interval of about 6 years. This event was the third largest flood of record. Both floods resulted in damages to homes and businesses adjacent to Corte Madera Creek. In December 1983, the Marin County Board of Supervisors requested the Corps to reinitiate the project.

In 1989, after extensive local coordination, engineering analysis of the data, and consideration of experience obtained from the recent floods, the U.S. Army Engineer District, Sacramento, which had design responsibility at that time, presented a new plan for Unit 4. This was the 1989 Sacramento District "selected plan" which included channel improvements, floodwalls, and a sediment trap. It provided protection from the one percent chance exceedance or 100-yr flood of 6,900 cfs at the Ross gauge and 8,800 cfs at San Francisco Bay. The plan consisted of a 270-ft extension of the concrete channel with a sloping concrete drop structure at its upstream end. An earthen trapezoidal channel extended about 350 ft upstream to the Lagunitas Road Bridge where it joined a 300-ft-long sediment trap. The sediment trap was dredged 4 ft into the creek bed. The Lagunitas Road Bridge and approaches were raised. Upstream from Lagunitas Road Bridge, for about 1,200 ft, floodwalls were constructed on both the right and left overbanks. Channel improvements through the remainder of the project reach consisted of gabions and crib walls, placement of riprap, and planting of vegetation in selected areas to protect the channel banks from erosion and to prevent undercutting of the floodwalls and channel banks. Channel wall heights in the existing concrete channel were increased where necessary to meet freeboard requirements. Increases in wall heights of up to 7 ft were requested. This plan did not receive consensus approval from the cities of Ross, Larkspur, and Kentfield.

Marin County and the Corps worked together to develop a plan for a reduced level of flood protection. This became the 1989 "locally preferred plan." Based on engineering analysis, the capacity of the existing Lagunitas Road Bridge was

determined to be 5,400 cfs. This discharge, which is the 3.3 percent chance exceedance or 30-yr flood, was advanced as the design discharge. The plan included extending the concrete channel 300 ft upstream from the existing terminus of the concrete channel and adding a sediment basin upstream from Lagunitas Road Bridge. Favorable features of this plan, compared to the 1989 Sacramento District "selected plan," are: a.) lower floodwalls along the concrete channel, b.) no bridge modifications, and c.) and less modification to the existing creek channel in Unit 4.

At this point in the planning process, responsibility for the planning and design of the Corte Madera Creek flood control project was transferred from the Sacramento District to the U.S. Army Engineer District, San Francisco. From the initial public meetings it became apparent that while the communities involved conceptually agreed to the 1989 "locally preferred plan," objections to specific project features remained. The Corps then began a new planning investigation to evaluate several alternatives based on the concept of the 1989 "locally preferred plan" in an attempt to build a community consensus plan.

Purpose of Numerical Model Study

The numerical model study was performed to evaluate sedimentation processes for several alternatives for the Unit 4 reach of Corte Madera Creek. The model was used to determine the effectiveness of each alternative in reducing deposition in the downstream concrete channel, especially at the peak of the 5,400 cfs design flood. The effect of accumulated sediment on channel roughness was evaluated. Alternative maintenance dredging plans were also evaluated.

A numerical model of Corte Madera Creek had been developed by the U.S. Army Engineer Waterways Experiment Station which has now become the Engineer Research and Development Center, in 1988 to study the 1989 Sacramento District "selected plan." Copeland and Thomas (1989) reported results of that study in "Corte Madera Creek sedimentation study." The numerical model developed for the 1989 study was used for this investigation. It was unnecessary to repeat numerical model adjustment and circumstantiation, as that conducted for the original study was sufficient for this application.

The adequacy of the channel wall heights in the existing concrete channel was critical to the study. The numerical model used in this study, HEC-6W, is primarily a sediment transport model and does not have the refinements of a backwater model such as HEC-2 or HEC-RAS for calculation of design water-surface elevations. However, this numerical model may be used to determine the extent of the accumulated sediment under various conditions and the effect of the deposit on channel roughness. The model can also be used to evaluate the relative effects of various alternatives on water-surface elevations.

2 Numerical Model

Model Description

The HEC-6W one-dimensional numerical sedimentation model was used to make predictions in this study. Mr. William Thomas initiated development of this computer program at the U.S. Army Engineer District, Little Rock, in 1967. Further development at the U.S. Army Engineer Hydrologic Engineering Center by Mr. Thomas produced the widely used HEC-6 generalized computer program for calculating scour and deposition in rivers and reservoirs. Additional modification and enhancement to the basic program by Mr. Thomas and his associates at the U.S. Army Engineer Research and Development Center (ERDC) led to the HEC-6W program currently in use. The program produces a one-dimensional model that simulates the response of the riverbed profile to sediment inflow, bed material gradation, and hydraulic parameters. The model simulates a series of steady-state discharge events and their effects on the sediment transport capacity at cross sections and the resulting degradation or aggradation. The program calculates hydraulic parameters using a standard-step backwater method assuming subcritical flow.

If the backwater calculations indicate transition to supercritical flow, then the program assigns critical depth for water-surface elevation, but assigns supercritical normal depth for determining hydraulic parameters for sediment transport.

HEC-6W is a state-of-the-art program for use in mobile bed channels. The numerical model computations account for all the basic processes of sedimentation: erosion, entrainment, transportation, deposition, and compaction of the bed for the complete range of particle sizes found in Corte Madera Creek. The model calculates aggradation and degradation of the streambed profile over the course of a hydrologic event. When applied by experts using good engineering judgment, the HEC-6W program will provide good insight into the behavior of mobile bed channels such as Corte Madera Creek.

Channel Geometry

The channel geometry for the numerical simulations in this study was based on data obtained for the 1989 numerical model investigation (Copeland and Thomas 1989). The numerical model extends from Station 166+00 near the

mouth of Corte Madera Creek at San Francisco Bay to Station 392+00, which is just downstream from the confluence of Ross Creek. The channel geometry was based on cross sections from HEC-2 backwater models provided by the Sacramento District in 1988. Initial bed elevations in the channel between Station 166+00 and the concrete-channel stilling basin at Station 319+05 accounted for some antecedent deposition. In the 1989 numerical model study it had been decided that assigning deposition equal to 40 percent of that measured in a 1984 bathymetric survey would be a reasonable initial condition. It was deemed unreasonable to require annual maintenance of the earthen channel to design elevations, but it is expected that when deposition reaches 40 percent of measured 1984 elevations, maintenance dredging would be preformed. It was assumed that the concrete channel would be free of any sediment deposits at the beginning of the numerical simulations. This is equivalent to assuming prior maintenance in the concrete channel. Channel geometry in the design reach (Unit 4) was based on a combination of cross sections provided in 1988 by the Sacramento District from a HEC-2 backwater model and supplemental data obtained from field surveys conducted in 1999 by the consulting firm Philip Williams and Associates (PWA).

Reach lengths between cross sections are generally greater in an HEC-6W model than in an HEC-2 model. Reach lengths in this model were generally about 2,000 ft in the earthen channel downstream from the concrete channel and about 300 ft through the concrete channel and design channel reaches. Cross-section locations are shown in Figure 1.

Hydrographs

Discharge hydrographs are simulated in the numerical model by a series of steady-state events. A hydrograph simulated by a series of steady-state events of varying durations is called a histograph. The duration of each event is chosen so changes in bed elevation from deposition or scour do not significantly change the hydraulic parameters during that event. However, at relatively high discharges, durations need to be short; time intervals as low as 20 min were used for the design histograph. At low discharges, the time interval may be extended. Time intervals up to two days were used in this study.

An annual histograph with an imbedded flood histograph was used to evaluate design alternatives. The design annual histograph was based on the 1982 water year hydrograph. The design flood was developed from the flood of record on Corte Madera Creek that occurred 3 - 5 January 1982. The reported hourly discharges during this period were adjusted to obtain the design peak discharge. In this study, the design peak discharge was 5,400 cfs. A flood histograph with a design peak discharge of 5,400 cfs was obtained by multiplying reported hourly discharges between January 3 and 5 by a factor of 0.7826. A "minimum plan" with a design peak discharge of 4,100 cfs was also evaluated. The design flood histograph for the "minimum plan" had the same shape as the January 3-5, 1982 flood with all hourly discharges reduced by a factor of 0.5857.

Discharge data were obtained from United States Geological Survey (USGS) gauge records for the "Corte Madera Creek at Ross" gauge, located at Station 379+50, which is about 250 ft upstream from the Lagunitas Road Bridge behind Ross City Hall. Mean daily discharges greater than 100 cfs between October 1, 1981, and September 30, 1982, were used to develop the design histograph. Sediment transport is negligible for discharges less than 100 cfs. In addition to mean daily flows, USGS reported five peak discharges greater than 1,000 cfs for water year 1982. Histograph events were adjusted to account for the increased sediment transport potential during high-flow events. Reported peaks for all but the January 1982 peak were assigned durations of 4 hr and the corresponding mean daily flow was reduced to maintain the same daily runoff volume. The 4-hr duration was chosen based on durations of peak flows from December 1955 and March 1983 flood hydrographs.

Estimates for average annual dredging were estimated using four 14-year-long stochastic hydrographs. The stochastic hydrographs were developed by randomly choosing annual histographs from the 1972-1986 historical record to form a 14-year sequential histograph. This period was chosen because the annual histographs were already available from the 1989 study. Using this short period of record is adequate for comparison studies, but the entire historical record should be used if actual dredging maintenance estimates are needed.

Currently, when flow exceeds 3,000 cfs, flow breaks out of the Corte Madera channel both upstream from Lagunitas Road Bridge and upstream of the existing concrete channel. The Sacramento District developed a breakout discharge-rating curve for each of these locations for the 1989 numerical model study (Figure 2). This rating curve was based on high-water marks and reported discharges from the January 1982 flood. For most alternatives evaluated in this study, flood flows were assumed to be contained in the channel without breakout. This requires flow containment structures, such as upstream floodwalls or significant lowering of water-surface elevations by channel excavation. The breakout rating curve was used to evaluate "existing" conditions. In these cases, flow was removed from the model downstream from the breakouts. Breakout flows return to the channel downstream of the stilling basin near the confluence with Tamalpais Creek.

Downstream tributary inflow hydrographs were included in the model for the adjusted January 1982 flood. Tributary inflows were determined by the Sacramento District from a 1988 storm reconstitution study. A conclusion of those studies was that tributary contributions to the flow at the Ross gauge would be insignificant at discharges less than 4,000 cfs. Tributary inflow points were at College Avenue, Tamalpais Creek, an unnamed tributary near Station 303+00, and Larkspur Creek. The design peak discharge downstream from each tributary is shown in the following tabulation:

Downstream	Station	Discharge (cfs)
Upstream boundary	392+00	5,400
College Avenue	335+06	5,440
Tamalpais Creek	317+10	5,610
Unnamed Creek	303+00	5,960
Larkspur Creek	244+00	6,890

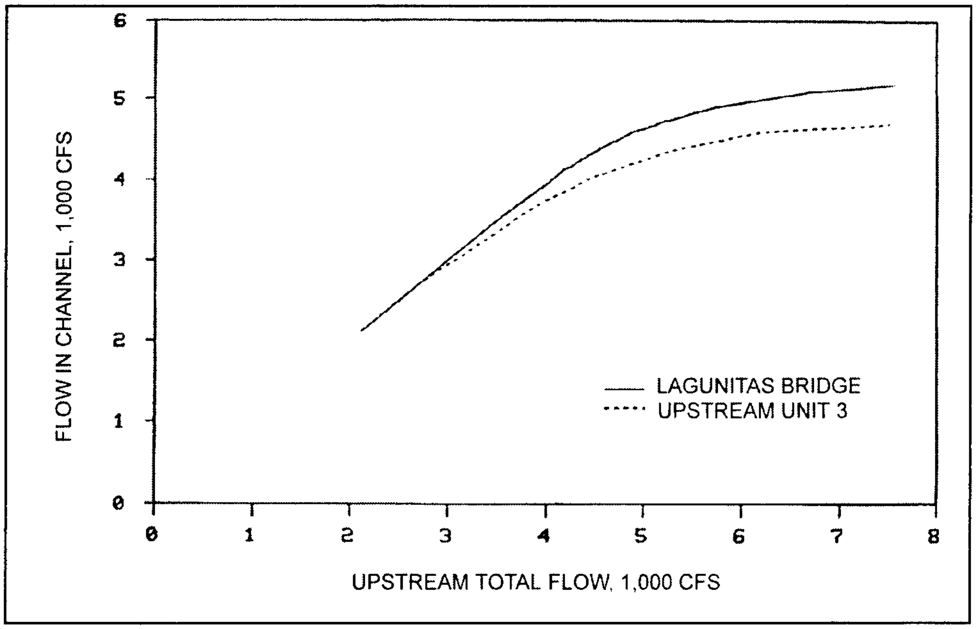

Figure 2. Breakout rating curve

Downstream Water-Surface Elevation

Starting water-surface elevations at the downstream end of the numerical model were determined using data from the National Oceanic and Atmospheric Administration (NOAA). The mean tide level at San Quentin (el 0.5) was assigned for most of the annual design histograph. This represents "expected" conditions. A higher downstream water surface elevation was assigned to the 3-day-long design flood portion of the annual histograph. The mean higher high water (mhhw) elevation used in the 1989 study was assigned to provide more severe (conservative) water-surface elevations and sediment deposition in the numerical model. The mhhw elevation for the 1960-72 epoch was el 2.9.

Bed Material

Initial bed-material gradations used in the numerical model are identical to those used in the 1989 numerical model study. The initial depths of the bed-sediment reservoir at each cross section are also identical to those used in the 1989 numerical model study.

The initial bed gradation for the reach downstream from the concrete channel was based on a sample collected 2,000 ft downstream from the stilling basin at Station 300+00 in April 1984. The median grain size of this sample was 0.2 mm. A single sample is generally insufficient to determine the gradation of a reach. However, in this case one sample was deemed adequate because (a) this reach is essentially a depositional reach and the gradation of the active layer will be determined from the inflowing sediment load instead of the bed sediment

reservoir; and (b) this reach was included in the numerical model primarily for the purpose of determining water-surface elevations at the downstream end of the concrete channel. Therefore accurate simulation of the bed profile was of secondary importance.

The bed-material gradation in the natural channel, upstream from the concrete channel, was determined from the average of four samples collected in January 1985 by the Sacramento District (Figure 3). These samples were taken between Station 372+00 (about 300 ft downstream from Lagunitas Road bridge) and Station 382+00 (about 250 ft upstream from the Ross gauge). This gradation was used to determine equilibrium sediment transport capacity at the upstream boundary of the numerical model as described in the section on sediment inflow. The depth of the initial bed-sediment reservoir was set equal to zero in this reach so that bed degradation below design inverts would not occur in the numerical model.

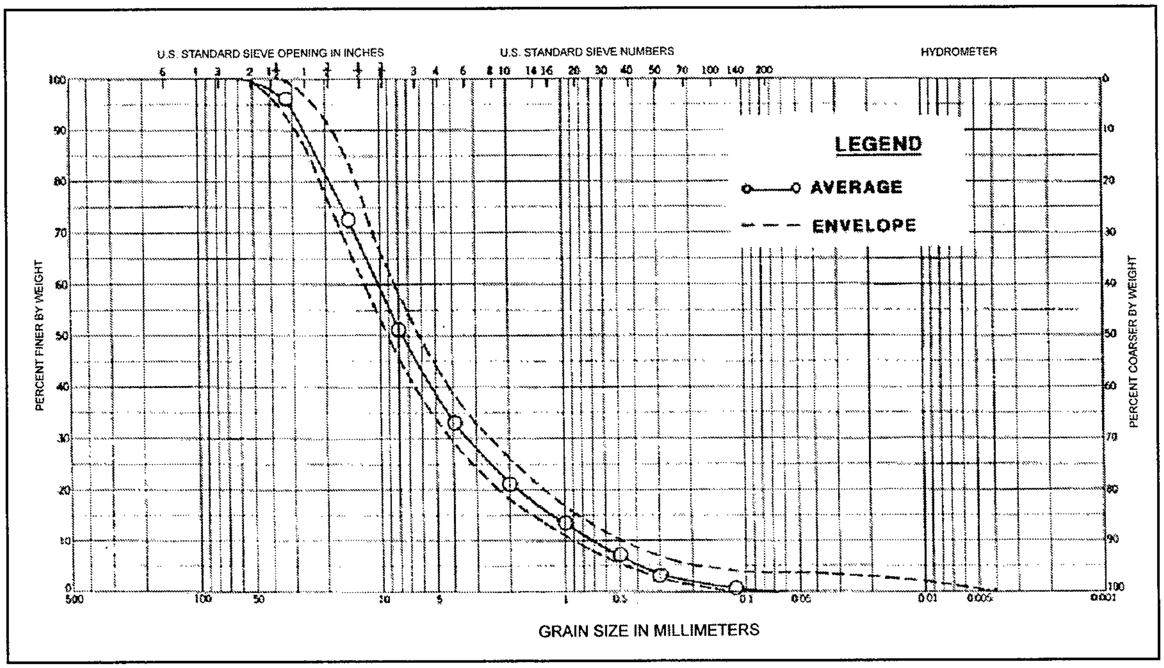

Figure 3. Bed-material gradations, vicinity of Ross gauge

Channel Roughness

Hydraulic roughness is influenced by grain size or bottom roughness, bank or sidewall roughness, bed form, water depth, changes in channel shape, and changes in the flow direction or distribution because of bends and confluences. In the one-dimensional numerical model these effects are accounted for by the Manning's roughness coefficient. Acceleration and deceleration of flow are accounted for with expansion and contraction coefficients. The roughness coefficient may vary significantly with discharge and time. The influence of grain or bottom roughness is known to decrease with increases in depth. On the

other hand, resistance can increase with increasing flow intensity if the size of bed forms travelling along the streambed also increase. Stream intensity can increase to the point where bed forms are washed out, in which case resistance declines dramatically. An attempt to account for these processes was made in this study by developing an algorithm that calculated composite roughness coefficients based on roughness attributed to the bed, sidewalls, and bed-load movement. The algorithm to calculate a composite hydraulic roughness coefficient at each cross section at every time-step in the numerical model was developed for the 1989 sediment study. This same algorithm was used in this study with slight modification.

Determining composite roughness in the concrete channel is complicated by the accumulation of sand and gravel on the channel bottom and aquatic growth on the channel sidewalls. A composite roughness coefficient was calculated using the following formula:

$$n = \left(\frac{P_{lw} n_{lw}^{1.5} + P_{uw} n_{uw}^{1.5} + P_b n_b^{1.5}}{P_{lw} + P_{uw} + P_b} \right)^{2/3}$$

where

n = Manning's roughness coefficient

P = perimeter

b = subscript denoting the bed

lw = subscript denoting the lower wall

uw = subscript denoting the upper wall

The wall roughness in the concrete channel varied depending on the presence of tube worms and barnacles. Tube worms and barnacles significantly increase the wall roughness as they protrude as much as 2 in. from the wall. Tube worm and barnacle growth exposed during a 1998 dredging operation is shown in Figure 4. In the numerical model the lower wall (below el 0.1) was assigned a roughness value of 0.021. The upper wall was assigned a roughness value of 0.014.

The bottom or bed roughness of the concrete channel in reaches where there was no sediment deposition was set equal to 0.017. This value is higher than normally assigned to concrete. The higher roughness assigned to the bed of Corte Madera Creek is attributed to concrete abrasion and to fish rests indented into the channel invert. Degraded channel invert conditions are shown in Figure 5.

Additional bed roughness occurs because of the movement of gravel bed load over the concrete surface. A relationship between gravel concentration and increase in bed roughness caused by gravel bed load transport was developed

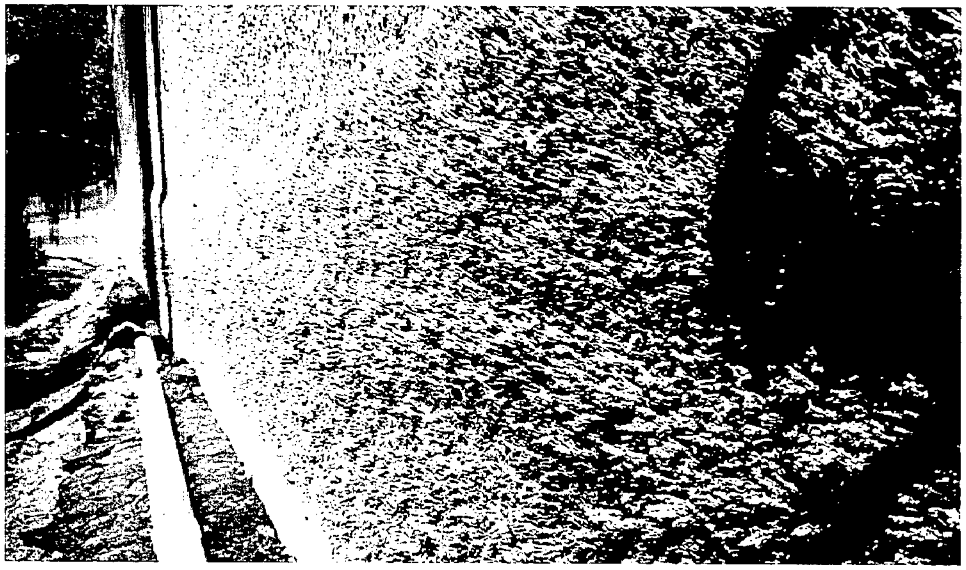

Figure 4. Tube worm and barnacle growth Station 323+00

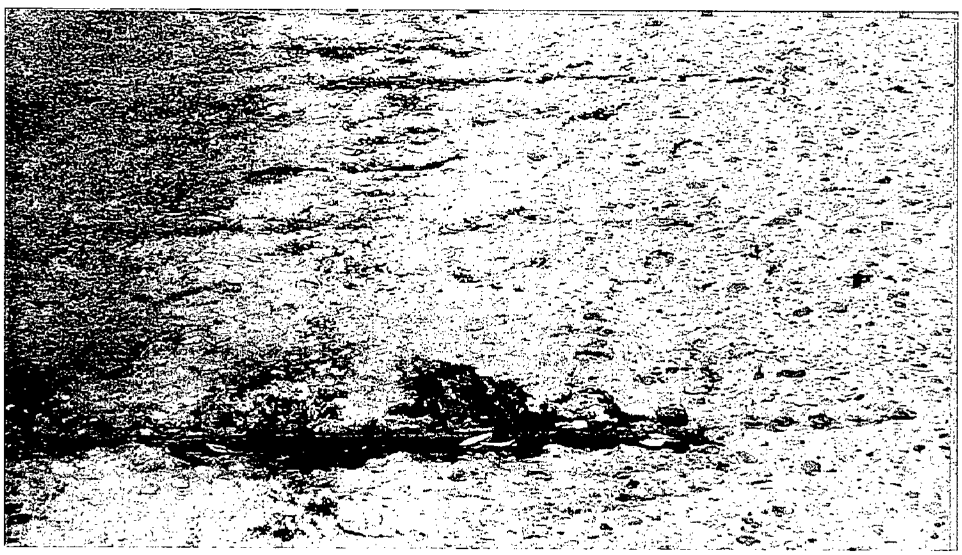

Figure 5. Degraded channel invert

from flume studies conducted at ERDC (Copeland, McVan, and Stonestreet 2000). The increase in Manning's roughness coefficient for the bed can be determined by the following regression equation:

$$n_{bedload} = 1.0312x10^{-6} C_{gravel}$$

where concentration, C, is in parts per million. Total bed roughness is the arithmetic sum of 0.017 and $n_{bed\ load}$. This equation applies as long as the bed load is moving along the bottom of the channel without deposition. Once deposition begins to occur, bed roughness increases significantly. Roughness caused by bed-load movement was not accounted for in the 1989 sediment study, but was added to the numerical model for this study.

When the numerical model calculated sediment deposition in the concrete channel, bed roughness was determined as the arithmetic sum of the grain roughness and bed-form roughness. The deposit had to attain an assigned minimum thickness before the grain and bed-form roughness algorithm was used by the model. The minimum thickness required in the model was the larger of 4 mm or two times the grain size for which the percent coarser fraction covered the bed to a thickness of two grain diameters.

The bed grain roughness was calculated using the Limerinos (1970) equation:

$$n_b = \frac{0.0926\left(R_b^{'}\right)^{1/6}}{1.16 + 2.03\log\left(\frac{R_b^{'}}{d_{84}}\right)}$$

where

$R_b^{/}$ = hydraulic radius of the bed attributed to grain roughness

d_{84} = particle size of which 84 percent of the bed is finer

Bed form roughness was accounted for in the numerical model based on calculations using the Brownlie (1983) equation for upper regime flow and hydraulic parameters from Corte Madera Creek for peak flow conditions. These calculations showed an increase in the Manning's bed roughness coefficient of 0.010 because of form roughness. In the numerical model, the bed roughness coefficient was increased to account for form roughness if both the minimum bed thickness and the critical shear stress were exceeded. The Shield's equation was used to determine critical shear stress:

$$\tau_c = \Theta(\gamma_s - \gamma_w)\, d_{50}$$

where

τ_c = critical shear stress

Θ = Shield's parameter

γ_s = specific weight of sediment

γ_w = specific weight of water

Various investigators have established a range for the Shield's parameter between 0.03 and 0.06 when median grain diameter is used in the equation. The following procedure was adopted to provide a continuous transition for the increase in roughness coefficient for form roughness. If the calculated shear stress was less than critical shear stress using a Shield's parameter of 0.03, then

the bed was assumed to be immobile and no adjustment was made to the Limerinos bed roughness. If the calculated shear stress was greater than the critical shear stress using a Shields parameter of 0.06, then the Limerinos bed roughness was increased by 0.010 to account for form roughness because of the mobile bed. The roughness increase was linearly interpolated for conditions between these limits.

Finally, hydraulic losses because of meandering were accounted for by using the Cowan (1956) meander adjustment factor. After a composite roughness coefficient was calculated for the bed and sidewalls, an adjustment factor of 1.15 was applied to the concrete channel cross sections. Cowan described this adjustment appropriate for channels with "minor" effects from meandering.

In the 1989 sediment study, high-water marks from the January 4, 1982 flood were used to evaluate the water-surface elevations calculated using the hydraulic roughness algorithm. In general, the calculated water-surface elevations in the HEC-6W model were slightly lower than the high-water marks. These water-surface comparisons are shown in Figure 6. Differences in reported and calculated values for the high-water marks taken along the channel may be attributed primarily to losses at bridges or to wave action which are not accounted for in the HEC-6W model. In addition, some of the high-water marks were taken a sufficient distance from the channel that they may have represented overbank conditions more than channel conditions. The assigned roughness coefficients were within the upper range used in engineering practice for this type of channel, and therefore, further increases in roughness coefficients were deemed unreasonable.

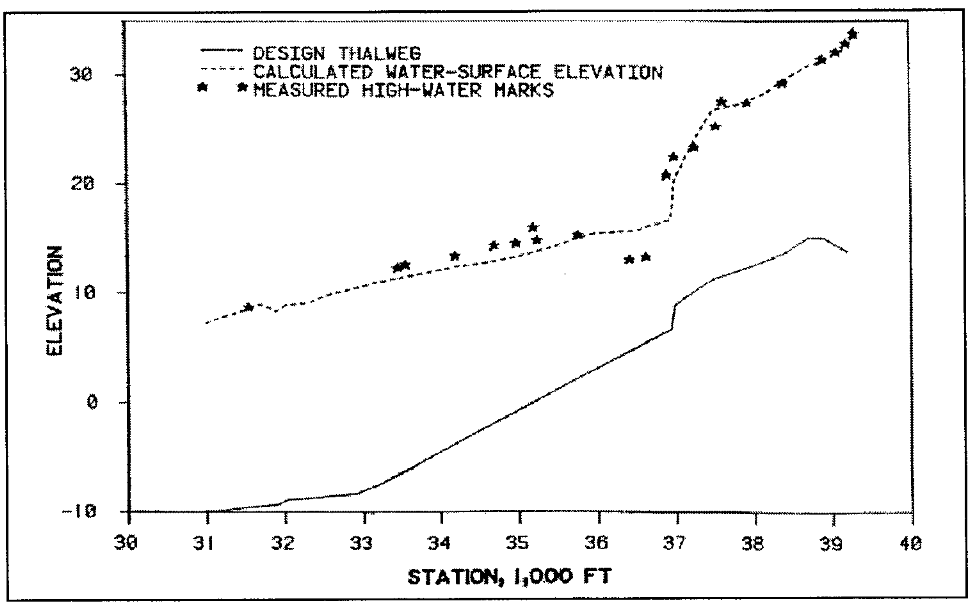

Figure 6. Maximum water-surface elevations, 4 January 1982

A roughness coefficient of 0.045 was assigned to the earthen channel downstream from the concrete channel. This is the same value used in the 1989 sediment study.

Roughness coefficients of 0.020 for the bed and 0.050 for the natural banks were used in the numerical model for the natural reaches upstream from the concrete channel. In reaches where the banks were modified as part of a specific design plan, a bank roughness of 0.045 was assigned in the numerical model. A composite channel roughness was calculated in the numerical model for the specific water depths at each time-step. The selected roughness coefficients in the natural reaches are considered to be at the lower end of the range used in engineering practice for this type of channel. These values were chosen to maximize sediment transport for the sediment study. Higher roughness coefficients should be used to determine water-surface elevations for flood control structures.

Sediment Inflow

The same sediment-inflow rating curves developed for the 1989 numerical model study were used for this study. The curves were developed using available measured suspended and bed-load data, calculations assuming equilibrium sediment transport, and measured gradations and volumes of sediment deposited in the concrete channel.

Measured sediment inflow data are inadequate to determine reliable sediment-inflow rating curves for the entire range of discharges and sediment sizes considered in this study. However, they were useful in defining the curves for discharges less than 2,000 cfs. The U.S. Geological Survey collected suspended sediment samples at the Ross gauge during 1978-1980. These samples were analyzed to determine particle-size distributions. Twenty-three events were reported with water discharges between 47 and 1,560 cfs. The highest measured flow was well below the design flood peak of 5,400 cfs. In addition to the suspended load measurements, seven bed-load samples were collected at the Ross gauge during the 1978 water year. Water discharges when the bed-load samples were collected varied between 47 and 1,180 cfs. Initial sediment-inflow rating curves for water discharges less than 2,000 cfs were developed based on optical fits of the measured data.

Initial sediment-inflow rating curves for discharges greater than 2,000 cfs were calculated using an equilibrium sediment transport equation. The Laursen-Copeland sediment transport function was used for the calculations. This modification of the Laursen (1958) equation was developed for use in the 1989 sedimentation study. The Laursen function is desirable because it was developed for size class analysis and considers parameters essential to both bed and suspended sediment loads. The Laursen-Copeland equation incorporated data for transport of gravels in addition to the sand data used to develop the original Laursen equation. The bed-material gradations used in the sediment transport equation were based on four samples collected in 1985 by the Sacramento District (Figure 3). These samples were collected in the channel reach between

750 ft downstream and 250 ft upstream of the Ross gauge. A more complete discussion of the Laursen-Copeland equation and its applicability to Corte Madera Creek is found in Copeland and Thomas (1989).

The sediment-inflow rating curves were adjusted during the adjustment phase of the 1989 study. With the adjustments, the numerical model was able to simulate historical deposition of sand and gravel in the concrete channel. Sediment inflow within the range of measured data (<1,500 cfs) was generally unchanged by the adjustments. The exception was very coarse and coarse gravel for which calculated transport did not agree with sampled transport; therefore, sediment inflow concentrations for these coarsest particles were based on equilibrium calculations in the numerical model. The justification for the adapted sediment inflow rating curves is based on successful simulation of gradations and volumes of measured deposition in the concrete channel.

The sediment inflow rating curves used in this study represent average conditions between 1972 and 1986. This is the historical period to which the numerical model was adjusted. Sediment inflow varies around this average under normal conditions and could be significantly greater with changes in the watershed. Significant increases in sediment inflow would be expected after a burn, a landslide, or untreated construction activity.

Model Adjustment and Circumstantiation

The numerical model was adequately adjusted and circumstantiated in the 1989 study, and there was no further adjustment for this study. No additional circumstantiation was possible because there have been no new data collection efforts.

The numerical model was adjusted and circumstantiated in 1989 to reproduce both the quantity and composition of historical sediment deposits in the concrete channel. The historical hydrograph between October 1972, when the existing concrete channel was completed, and May 1986 was simulated in the model. Surveys of channel deposition were conducted in July 1982, August 1984, January 1986, and May 1986. Bed material gradations were collected from the deposited material in August 1984, March 1986, May 1986, May 1987, and September 1987. The July 1982 and August 1984 data were used to adjust the numerical model. The 1986 and 1987 data were used to circumstantiate the numerical model.

A summary of the results of the 1989 circumstantiation study is reproduced here. Calculated deposition profiles after a numerical simulation of the historical hydrograph between October 1972 and January 1986 is compared to the measured bed profile in Figure 7. Accumulated deposition volumes are compared in Figure 8. These figures show that the numerical model reproduced an accurate profile downstream from College Avenue (Station 334+76), but underestimated deposition upstream. The model was very successful in predicting the quantity of total deposition in the channel during this period.

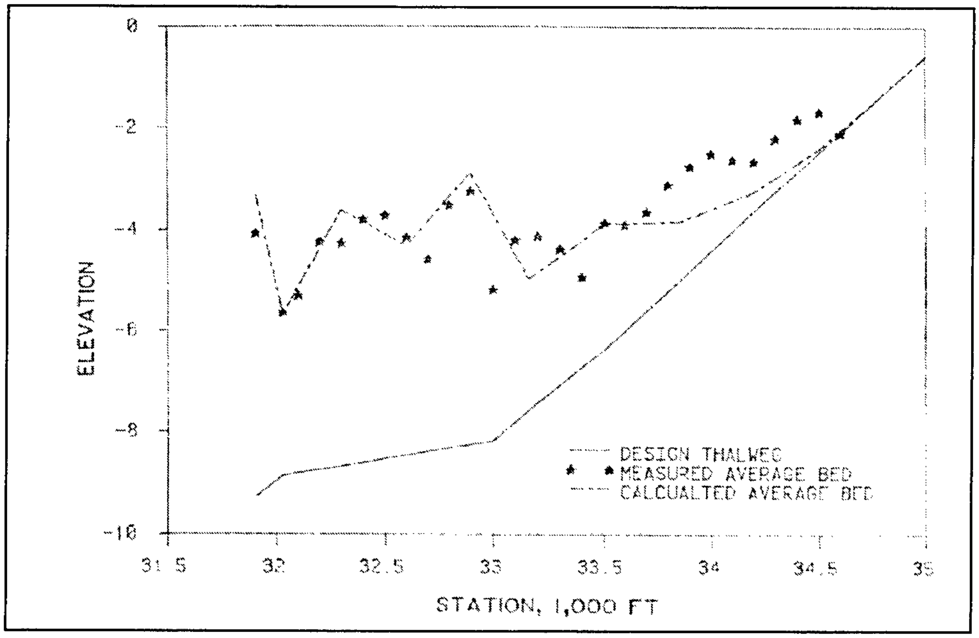

Figure 7. Aggradation in concrete channel, October 1972 to January 1986

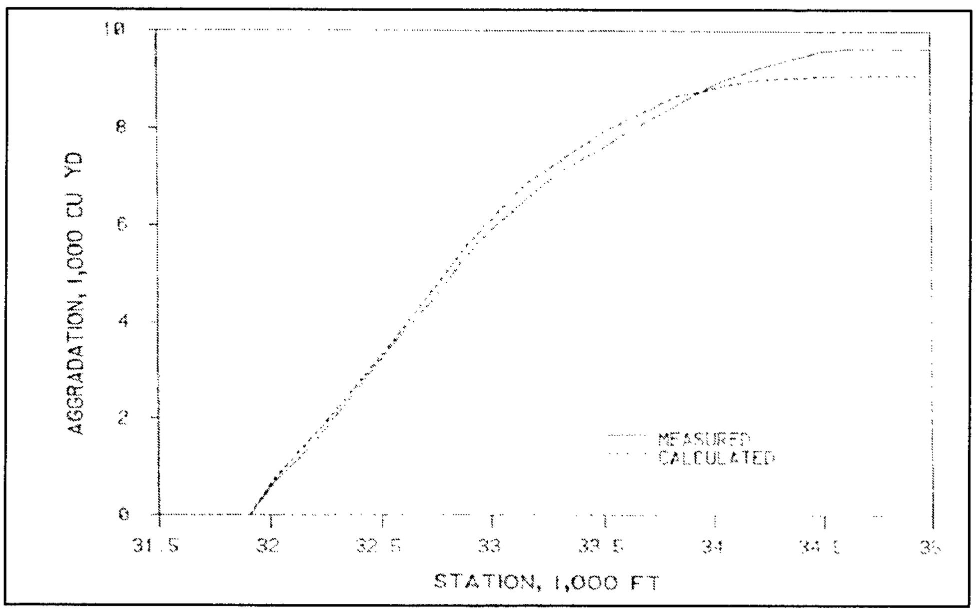

Figure 8. Accumulated aggradation in concrete channel, October 1972 to January 1986

A significant runoff event occurred in February 1986, when an estimated peak discharge of 4,150 cfs occurred at the Ross gauge. A deposition survey in the concrete channel was taken in May 1986. Calculated and surveyed deposition profiles for the October 1972-May 1986 simulation are compared in Figure 9. Changes in accumulated deposition in the concrete channel are shown in Figure 10. Based on field surveys, about 2,800 cu yd of material were removed from the concrete channel during the February 1986 flood. This

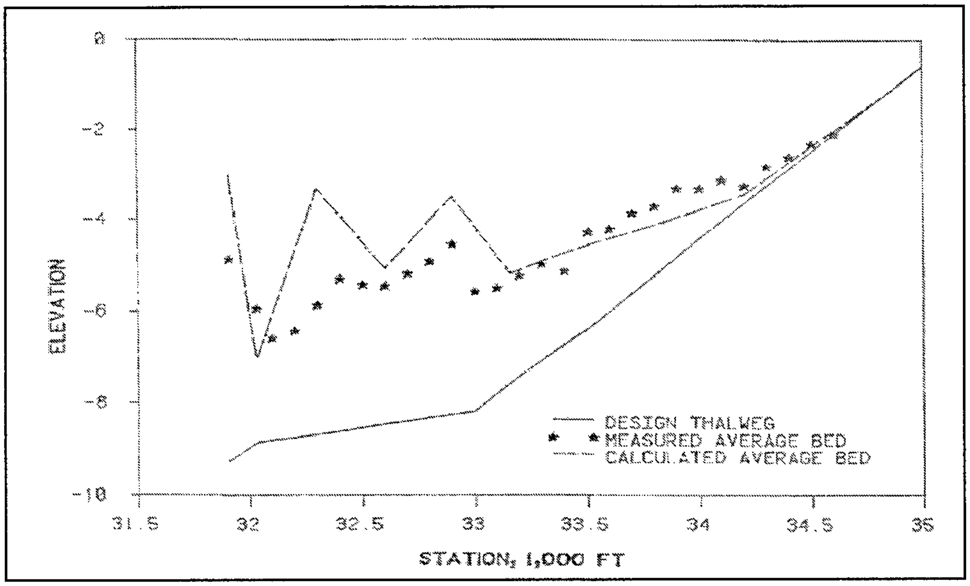

Figure 9. Aggradation in concrete channel, October 1972 to May 1986

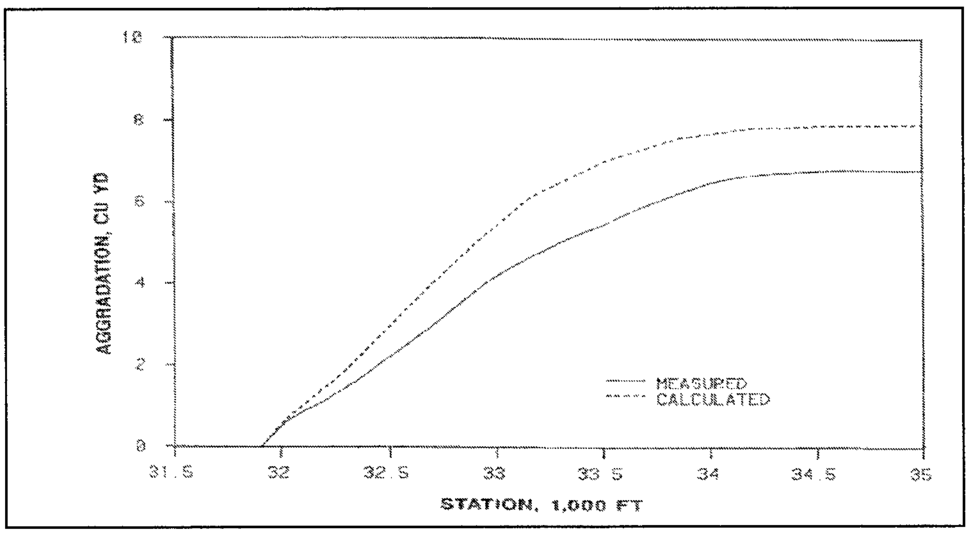

Figure 10. Accumulated aggradation in concrete channel, October 1972 to May 1986

compares to a removal of about 1,200 cu yd calculated in the model. This result was consistent with the comparison of field surveys and calculated deposition profiles after the flood of January 1982, which also showed more material removed in the prototype. These results indicate that the model, using average sediment inflow rating curves, underestimates the ability of flood flows to remove deposited sediment from the concrete channel. Using numerical model results will therefore produce conservative design estimates in terms of sediment removal by high flows.

In May 1987, bed-material samples were collected at 15 locations in the concrete channel between Stations 326+00 and 337+50. The numerical

simulation ended in March 1986, but the period between March 1986 and May 1987 was a period of relatively low runoff. During this period there were only five days where the mean daily flow exceeded 100 cfs. Because of the small amount of runoff between March 1986 and the collection of the bed-material samples in 1987, it was deemed reasonable to compare these measurements with calculated gradations. Calculated and sampled gradations are compared in Figure 11.

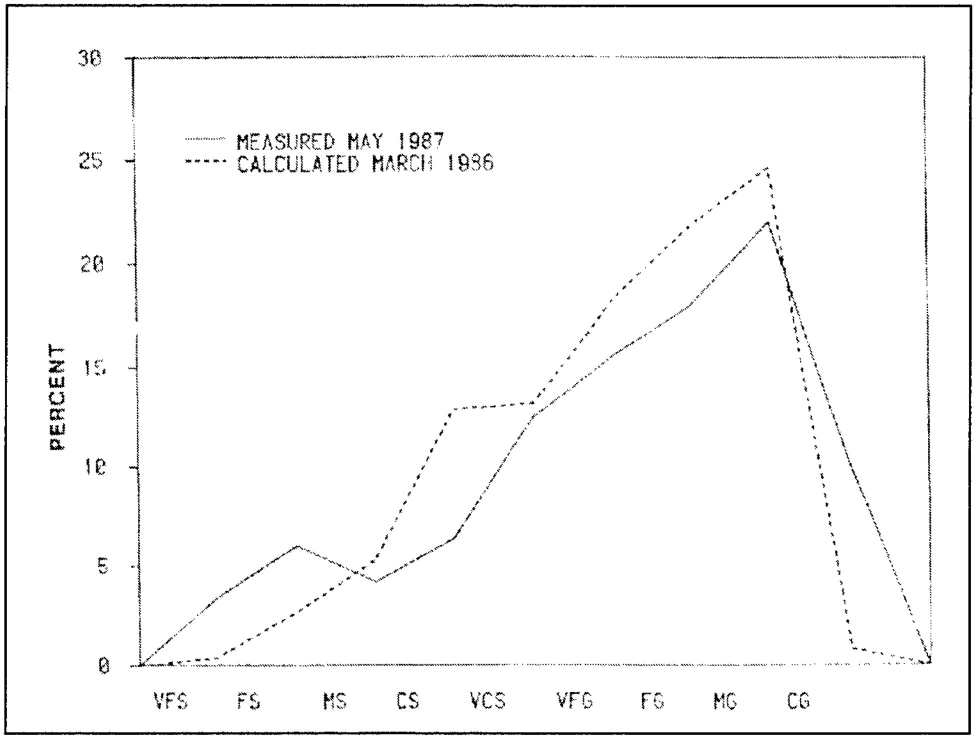

Figure 11. Bed-material gradation between stilling basin and College of Marin Bridge

Numerical model circumstantiation demonstrated that the model could be used to evaluate the proposed improvements for Unit 4 on Corte Madera Creek. It is recognized that reliability of model predictions is limited because of the uncertainty related to sediment inflow and to bed-material gradation data. The model was successful in simulating the longitudinal extent and the quantity of deposition in the concrete channel. Bed-material gradations were reproduced fairly accurately. These gradations are important because they influence the roughness of the channel bed. The model predicted degradation during flood events. Because predicted quantities of degradation were less than the measured quantities, the model will provide conservative results with respect to the ability of the channel to maintain a sediment-free bed.

3 Study Results

Study Approach

The numerical sedimentation study was conducted to support the general planning study for Corte Madera Creek. Initially, the sedimentation effects of various alternatives were evaluated simply by comparing calculated sediment deposition in the concrete channel and in the proposed sediment basins for the design annual hydrograph. As the study progressed and alternative designs were fine-tuned, more detailed sediment analyses were performed. These included calculating roughness coefficients in the concrete channel and evaluating alternatives with various maintenance schemes.

One objective of the planning study was to obtain a consensus plan that would be accepted by the community. Therefore, alternatives that did not receive consensus support in the planning process were not analyzed in detail in the sedimentation study. For example, the 1989 Sacramento District "selected plan," which is labeled the Type 3 Design in this study, performed well with respect to sedimentation processes, but was not evaluated in detail because of its lack of consensus community support.

Existing Conditions

Sediment accumulation in the existing mild-sloped portion of the concrete channel for the design annual hydrograph with existing conditions was calculated using the numerical model. These results were used to compare alternative plans evaluated during the course of the study.

Although there is no constructed sediment basin in the existing channel at the Lagunitas Road Bridge, the channel is relatively wide at this location and occasional channel excavation has historically occurred here. In recent years, this excavation has been undertaken annually by the town of Ross. This location is considered to be a "natural" sediment basin. The cross sections initially used in the numerical model were based on a 1988 HEC-2 model developed by the Sacramento District. The cross sections in that model were most likely surveyed after such a channel excavation; therefore the numerical model calculates sediment deposition at this location even without a constructed sediment basin. When the 1999 cross sections from the Philip Williams and Associates field

survey were incorporated into the numerical model, calculated sediment deposition in the "natural" sediment basin was less. This is attributed to the higher bed elevations that existed at the time the PWA survey was conducted.

Currently, when the discharge exceeds 3,000 cfs on Corte Madera Creek, flows break out of the channel upstream from Lagunitas Road Bridge and at the upstream terminus of the concrete channel. When flow breaks out of the channel it carries very little of the coarse sediment load. This is because the coarse load is carried primarily toward the bottom of the water column (Rouse 1937), and water breaks out over the banks at the top of the water column. The consequence of this natural process is an increase in the concentration of coarse load and an increase in sediment deposition in the concrete channel downstream.

The 1989 "locally preferred plan," which had a design discharge of 5,400 cfs, included floodwalls throughout the Unit 4 reach. These floodwalls would provide containment of design flows in Corte Madera Creek. An alternative evaluated here contained flows up to 5,400 cfs with no modification to the existing channel. This is considered herein as "existing conditions with containment." This is the plan to which alternatives were compared to in this study.

The with-containment and without-containment alternatives for the existing channel were evaluated using the 5,400-cfs design annual histograph. The starting water surface was set at el 2.9 during the 3-day-long design flood and at el 0.5 for the remainder of the year.

Accumulated deposition in the mild-sloped portion of the concrete channel at the peak of the design flood and at the end of the one-year simulation was calculated using the numerical model. Deposition in the concrete channel at the peak of the flood is important because deposition increases channel roughness and therefore water depths, which then require increased wall heights in the concrete channel. Deposition at the end of the one-year simulation is useful in determining maintenance dredging requirements. Accumulated deposition for the one-year simulation for the existing channel is shown in the following tabulation.

Accumulated Deposition in Concrete Channel with Design Flood One-Year Simulation (cu yd)		
Plan	**Peak of Design Flood**	**End of Year**
Existing	879	4,466
Existing with upstream containment	58	4,651

Significantly less sediment is deposited in the concrete channel at the peak of the design flood when flows are contained upstream. Roughness coefficients for the design flood will be significantly less with upstream containment. Upstream containment therefore is one of the most important features of any flood control scheme on Corte Madera Creek.

Type 1 and Type 2 Designs - Sediment Basins and Floodwalls

The initial plans for Unit 4 of Corte Madera Creek evaluated in this study consisted of excavating two small sediment basins into the natural creek bed upstream and downstream from Lagunitas Road Bridge. Basically, these sediment basins would be an enlargement of the existing "natural" sediment basin. The existing stream banks would not be disturbed and the existing channel side slopes would be maintained throughout the length of the sediment basins. No other channel modifications were associated with these plans. A 200-ft-long upstream sediment basin extended between Stations 378+48 and 376+50. A 150-ft-long downstream basin extended between Stations 375+00 and 373+47. The Type 1 Design called for 3 ft of excavation below the existing channel bed. The Type 2 Design called for 5 ft of excavation below the existing channel bed in both sediment basins. Profiles for the Type 1 sediment basins and representative cross sections for the Type 1 and Type 2 sediment basins are shown in Figures 12-15. These sediment basins would be excavated to design bed elevations every year.

These plans were evaluated assuming that the 5,400 cfs design discharge would be contained upstream from Lagunitas Road Bridge. This assumption requires the inclusion of floodwalls into the Type 1 and Type 2 Designs.

These two alternatives were evaluated using the 5,400-cfs design annual histograph. The downstream water surface was set at el 2.9 for the 3-day-long design flood and at el 0.5 for the remainder of the year.

Both plans were evaluated by comparing accumulated deposition in the concrete channel and in the sediment basins at the peak of the design flood and at the end of the one-year simulation. Accumulated deposition for the one-year simulation with the Type 1 and Type 2 designs is compared to deposition under existing conditions with upstream containment in the following tabulation.

Accumulated Deposition with Design Flood – One-Year Simulation (cu yd)				
	Peak of Design Flood		End of year	
Plan	Sediment basins	Concrete channel	Sediment basins	Concrete channel
Existing with upstream containment	1,116	58	770	4,651
Type 1	1,731	32	1,321	4,835
Type 2	1,908	29	1,901	4,563

The tabulation shows sediment basin deposition for existing conditions with upstream containment even though there is no constructed sediment basin with this alternative because sediment deposits naturally in this reach. The 1988 Sacramento District cross sections were used in these comparisons.

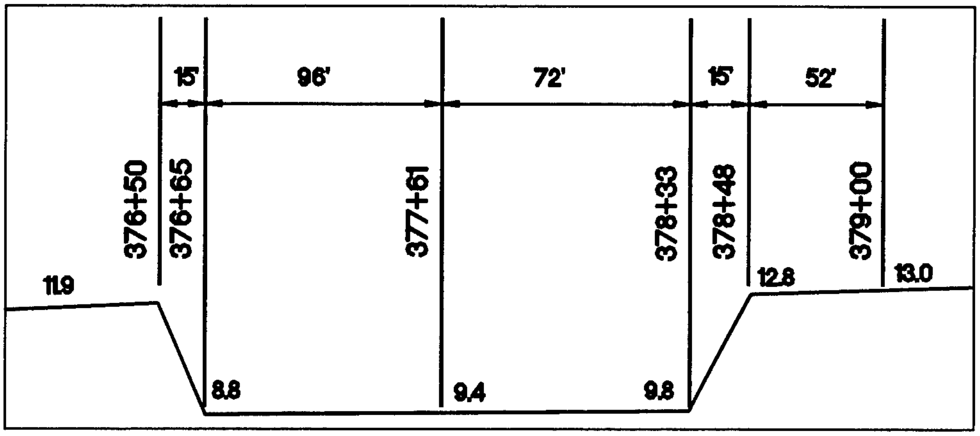

Figure 12. Type 1 upstream sediment basin profile

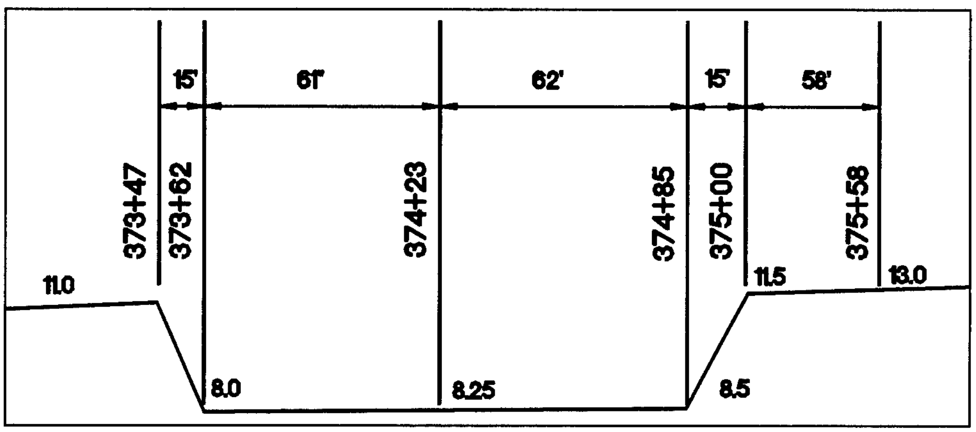

Figure 13. Type 1 downstream sediment basin profile

The Type 1 and Type 2 Designs provide for a decrease of about 26 and 29 cu yd, respectively, in the concrete channel at the peak of the design flood. This will result in a slight decrease in channel roughness coefficients for determining wall heights. As expected, sediment accumulations in the sediment basin at the end of the year are significantly greater with the Type 1 and Type 2 Designs. With the Type 1 and Type 2 Designs, significant sediment continues to accumulate in the concrete channel by the end of the year.

The small sediment basins in the Type 1 and Type 2 Designs provide little additional benefit when compared to the existing condition with upstream containment.

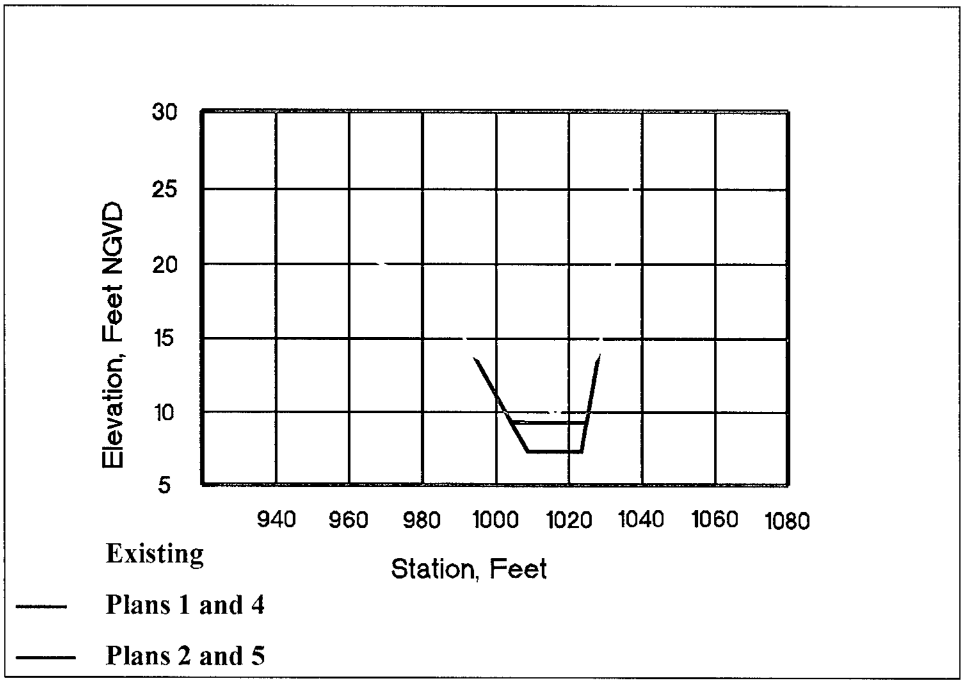

Figure 14. Type 1 and Type 2 upstream sediment basin cross section at Station 377+61

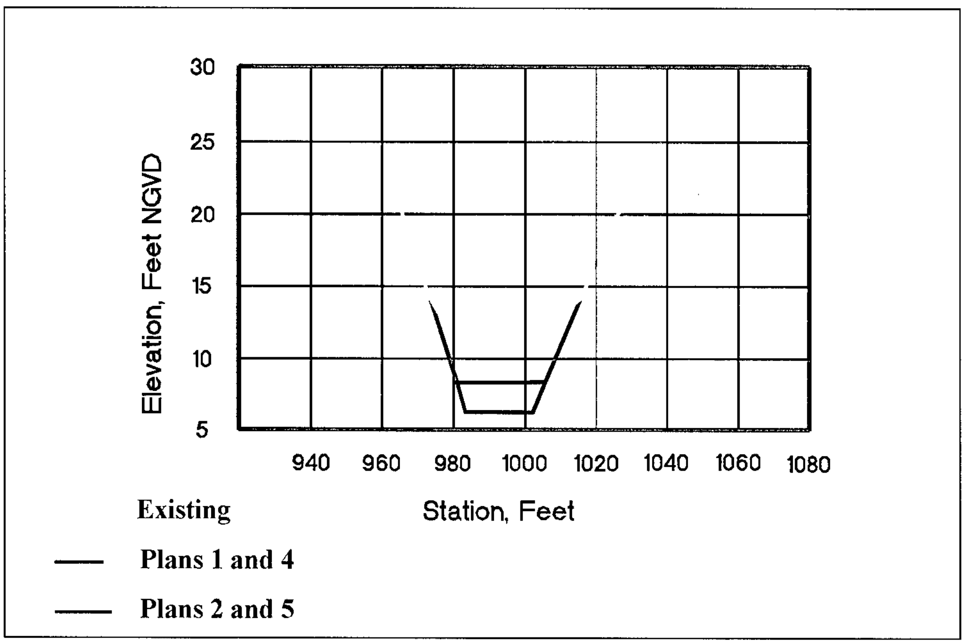

Figure 15. Type 1 and Type 2 downstream sediment basin cross section at Station 374+23

Type 3 Design - 1989 Sacramento District "Selected Plan"

The Type 3 Design is the 1989 Sacramento District "selected plan." This plan was evaluated to see how it would function with the reduced design annual histograph.

The "selected plan" provided for a 270-ft upstream extension of the concrete channel with a sloping concrete drop structure at its upstream end. An earthen trapezoidal channel extended about 350 ft upstream to the Lagunitas Road Bridge where it joined a 300-ft-long sediment trap. The sediment trap was dredged 4 ft into the creek bed. The Lagunitas Road Bridge and approaches were raised. Upstream from Lagunitas Road Bridge, for about 1,200 ft, floodwalls were constructed on both the right and left overbanks containing the design flow.

This alternative was evaluated using the design annual histograph. The downstream water surface was set at el 2.9 for the 3-day-long design flood and at el 0.5 for the remainder of the year.

Accumulated deposition for the one-year simulation with the Type 3 Design is compared to deposition under existing conditions with upstream containment in the following tabulation.

Accumulated Deposition with Design Flood – One-Year Simulation (cu yd)				
	Peak of Design Flood		End of Year	
Plan	Sediment basin	Concrete channel	Sediment basin	Concrete channel
Existing with upstream containment	1,116	58	770	4,651
Type 3	5,480	13	5,367	4,572

The calculations show that the Type 3 Design provided for a decrease of about 45 cu yd (or 77 percent) of sediment deposition in the concrete channel at the peak of the design flood. This translates to lower roughness coefficients for determining concrete channel wall heights. As expected there is significantly more sediment to be removed from the sediment basin each year. The larger sediment basin did not significantly decrease the quantity of sediment deposited in the concrete channel by the end of the year.

This plan did not receive more attention because it would require significant disturbance of the existing natural stream.

Type 4 and Type 5 Designs - Extended Upstream Sediment Basin and Floodwalls

The Type 4 and Type 5 Designs called for extending the upstream sediment basin from the Type 1 and Type 2 Designs about 560 ft to Station 384+09. This

made the upstream sediment basin about 760 ft-long. The Type 4 Design called for 3 ft of excavation below the existing channel bed. The Type 5 Design called for 5 ft of excavation below the existing channel bed. The rest of the channel in the Unit 4 reach was unmodified.

These plans were evaluated assuming that the 5,400 cfs design discharge would be contained upstream from Lagunitas Road Bridge. This assumption requires the inclusion of floodwalls into the Type 4 and Type 5 Designs.

These two alternatives were evaluated using the design annual histograph. The downstream water surface was set at el 2.9 for the 3-day-long design flood and at el 0.5 for the remainder of the year.

The plans were evaluated by comparing accumulated deposition in the concrete channel and in the sediment basins at the peak of the design flood and at the end of the one-year simulation. Accumulated deposition for the one-year simulation with the Type 4 and Type 5 Designs is compared to deposition under existing conditions with upstream containment in the following tabulation.

Accumulated Deposition with Design Flood – One-Year Simulation (cu yd)				
	Peak of Design Flood		End of Year	
Plan	Sediment basins	Concrete channel	Sediment basins	Concrete channel
Existing with upstream containment	1,116	58	770	4,651
Type 4	1,875	28	1,906	4,519
Type 5	2,067	22	3,162	3,602

The Type 4 and Type 5 sediment basins did not significantly reduce sediment deposition in the concrete channel at the peak of the design flood when compared to the Type 1 and Type 2 sediment basins. With the Type 5 Design more annual sediment deposition occurred in the sediment basins and less in the concrete channel than it did with the Type 1, Type 2 or Type 4 Designs.

The additional 560 ft of disturbance to the existing Corte Madera Creek channel required for the Type 4 and Type 5 Designs made these alternatives unattractive, and they were dropped from further consideration.

Type 6 and Type 7 Designs - Downstream Channel Excavation and Upstream Sediment Basin

The floodwalls required to contain the design flood with the previously evaluated alternatives were deemed undesirable. To lower the water-surface elevations channel modifications downstream from Lagunitas Road Bridge were considered.

The Type 6 Design consisted of deepening the existing channel about 5 ft between Stations 369+50 and 375+88. This created a channel invert between

el 6.0 and 6.5, which is lower than the invert of the existing concrete channel at its upstream terminus (el 6.83). Channel excavation would be conducted so the existing streambanks would not be disturbed and the existing channel side slopes would be maintained. The Type 6 Design also included a 5-ft-deep sediment basin upstream from Lagunitas Road Bridge which was the same size as the upstream basin in the Type 2 Design.

The Type 7 Design consisted of a concrete transition channel and a widened earth channel between Stations 369+50 and 375+88. It was the channel designed by the Sacramento District as part of the 1989 Sacramento District "selected plan." This plan eliminated channel constrictions responsible for increased water-surface elevations. The Type 7 Design also included a 5-ft-deep sediment basin upstream from Lagunitas Road Bridge. This basin was the same size as the upstream basin in the Type 2 Design.

Water-surface elevations were calculated using the HEC-6W numerical sedimentation model to provide a general idea of how the Type 6 and Type 7 Designs would affect water surfaces between the upstream terminus of the concrete channel (Station 369+50) and Lagunitas Road Bridge (Station 375+88). The HEC-6W calculations should be used for comparative purposes only. This model does not account for bridge losses, and the roughness coefficients used in the numerical model were chosen to maximize sediment transport, not to predict water-surface elevations for floods. Calculated water-surface elevations for the Type 6 and Type 7 Designs are compared to calculated water-surface elevations for the existing channel with containment in Figure 16. It shows the top of bank (TOB) that was estimated from 2-ft contour interval topographic mapping and should be considered approximate. Figure 16 demonstrates that the Type 6 Design actually causes about a 0.5-ft increase in water-surface elevations. The Type 7 Design is effective in reducing water-surface elevations. Calculated water-surface elevations with the Type 7 Design are near existing top of bank elevations, which eliminates the need for high floodwalls downstream and upstream from Lagunitas Road Bridge.

Although the objective of these plans was to eliminate the need for floodwalls, calculations indicated that the inclusion of floodwalls would still be required with the Type 6 Design.

The Type 6 and Type 7 Designs were evaluated using the design annual histograph. The downstream water surface was set at el 2.9 for the 3-day-long design flood and at el 0.5 for the remainder of the year.

The plans were evaluated by comparing accumulated deposition in the concrete channel and in the sediment basins at the peak of the design flood and at the end of the one-year simulation. Accumulated deposition for the one-year simulation with the Type 6 and Type 7 Designs is compared to deposition under existing conditions with upstream containment in the following tabulation.

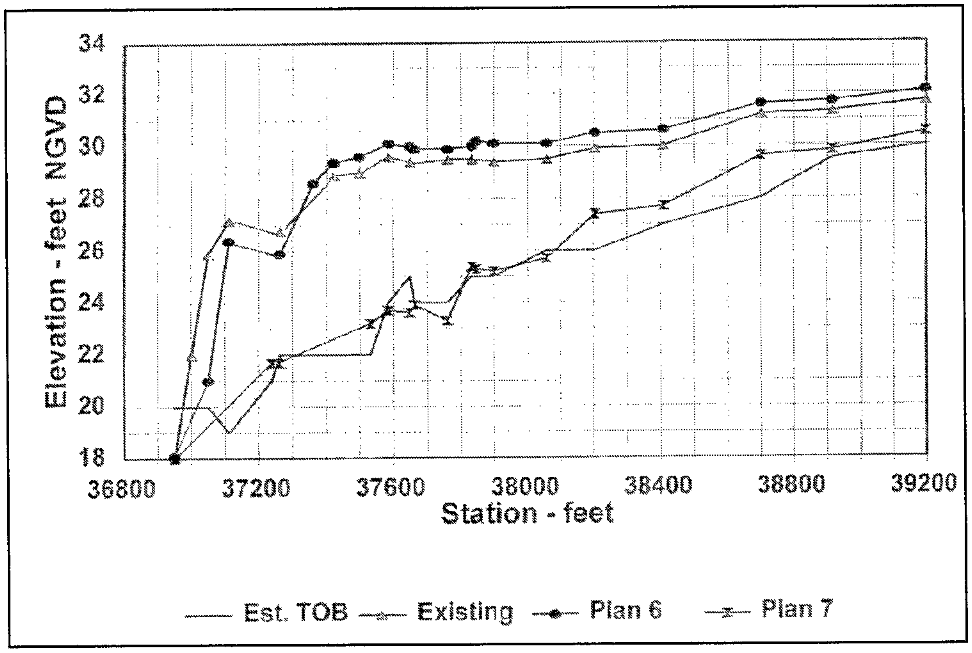

Figure 16. Calculated water-surface elevations for Type 6 and Type 7 Designs

Accumulated Deposition with Design Flood – One-Year Simulation (cu yd)				
	Peak of Design Flood		End of Year	
Plan	**Sediment basin**	**Concrete channel**	**Sediment basin**	**Concrete channel**
Existing with upstream containment	1,116	58	770	4,651
Type 6	1,764	24	830	3,995
Type 7	11	40	134	2,702

The sediment deposits in the concrete channel during the peak flow and at the end of the design hydrograph were on the same order of magnitude with Type 6 Design as with the previously evaluated sediment basin plans. With the Type 7 Design, however, the sediment basin was ineffective in terms of capturing any sediment before the peak of the design flood. The reason was attributed to increased velocities upstream from Lagunitas Road Bridge because of lower downstream water-surface elevations. However, deposition of sediment in the concrete channel at the peak of the design flood was 40 cu yd, slightly greater than the previous sediment basin alternatives and less than with existing conditions.

Type 8, Type 9, and Type 10 Designs - San Francisco District Proposal for Downstream Channel Excavation and Upstream Sediment Basin

The San Francisco District proposed widening and deepening the existing channel between Stations 369+50 and 373+75 and excavating a sediment basin

between Stations 373+75 and 377+61. Channel constrictions present in the Type 6 Design were removed, but widening was less than with the Type 7 Design. Generally, these plans called for a trapezoidal channel with 1V: 1H side slopes. The left descending bank had a vertical retaining wall between Stations 369+50 and 373+75. The right bank retained its existing bank and side slope.

The Type 8 Design consisted of deepening and widening the existing natural channel from the end of the concrete channel at Station 369+50 to Station 377+61, about 800 ft. The channel was excavated to el 6.5. The base width of the excavated channel was 30 ft between Stations 369+50 and 373+25, increasing in width to 35 ft at Station 373+75. The base width widened to 45 ft at Station 374+50, to 55 ft at Station 375+50 and finally to 65 ft between Stations 375+80 and 377+61. The Type 8 Design did not include an excavated sediment basin.

The Type 9 Design had the same channel enlargement dimensions as the Type 8 Design, but included an excavated sediment basin between Stations 373+75 and 377+61. The sediment basin for the Type 9 Design was excavated to el 4.5. This excavation resulted in reduced channel base widths in the sediment basin reach compared to the Type 8 Design.

The Type 10 Design was the same as the Type 9 Design, except that the sediment basin was excavated an additional 3 ft to el 1.5.

These plans were evaluated assuming that the 5,400 cfs design discharge would be contained upstream from Lagunitas Road Bridge. This assumption may require the inclusion of floodwalls in the Unit 4 reach.

The Type 8 - 10 Designs were evaluated using the design annual histograph. The downstream water surface was set at el 2.9 for the 3-day-long design flood and at el 0.5 for the remainder of the year.

The plans were evaluated by comparing accumulated deposition in the concrete channel and in the sediment basins at the peak of the design flood and at the end of the one-year simulation. Accumulated deposition for the one-year simulation with the alternative designs is compared to deposition under existing conditions with upstream containment in the following tabulation.

Accumulated Deposition with Design Flood – One-Year Simulation (cu yd)				
	Peak of Design Flood		**End of Year**	
Plan	**Sediment basin**	**Concrete channel**	**Sediment basin**	**Concrete channel**
Existing with upstream containment	1,116	58	770	4,651
Type 8	2,644	21	2,101	2,952
Type 9	4,053	14	3,843	2,577
Type 10	5,515	9	6,400	2,005

All three plans effectively reduced sediment accumulation in the concrete channel for the peak discharge. The Type 9 and Type 10 Designs were

comparable to the 1989 Sacramento District "selected plan" (Type 3) in terms of sediment accumulation at the peak. In addition, the Type 10 Design sediment basin provided sediment trap efficiency comparable to that of the 1989 Sacramento District "selected plan." All three plans had less sediment accumulation in the concrete channel by the end of the year when compared to the Types 1, 2, and 4-7 sediment basin alternatives.

Calculated water-surface elevations (Figure 17) were lower than those calculated for the Type 6 Design, but higher than the Type 7 Design. Floodwalls would still be required upstream of Station 369+50 for these three plans.

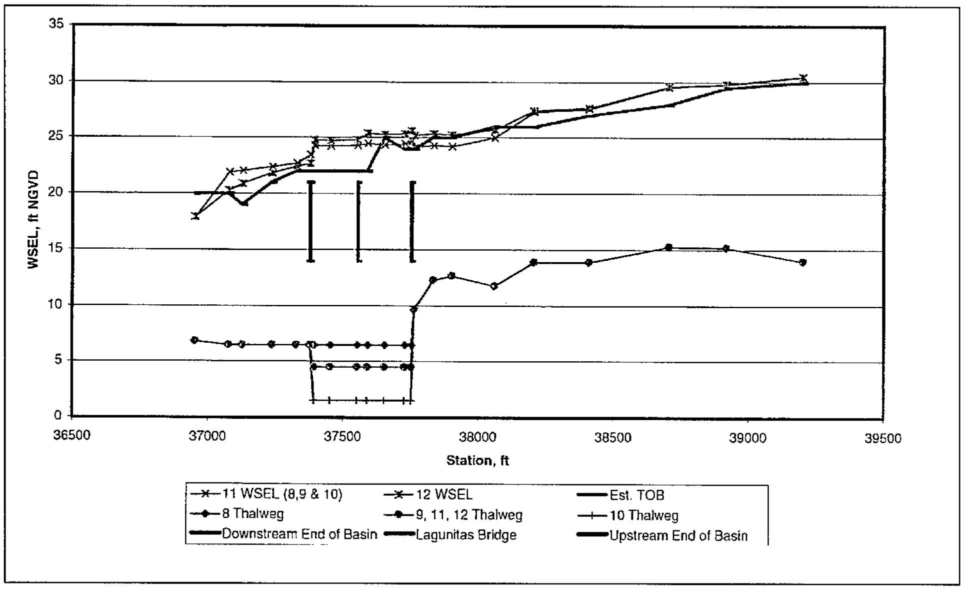

Figure 17. Calculated water-surface elevations for Type 8-12 Designs

Type 11, Type 12, and Type 13 Designs - Reduced Downstream Channel Excavation and Upstream Sediment Basin

The next series of channel designs called for reducing the width of the proposed sediment basin and less channel modification between the upstream terminus of the concrete channel and the proposed sediment basin.

The Type 11 Design was the same as the Type 9 Design, except that the width of the sediment basin upstream from Lagunitas Road Bridge between Station 375+80 and 377+61 was reduced from 60 ft to 30 ft. This allowed for retention of an existing bank fill on the left bank upstream from Lagunitas Road Bridge.

The Type 12 Design had the same sediment basin as the Type 11 Design. In addition, the Type 12 Design called for a channel base width widening of 25 ft between Stations 369+50 and 373+25 which is 5 ft less than in the Type 8-11

Designs. The Type 12 Design also called for a channel base width widening of only 30 ft between Stations 373+25 and 375+50, which is between 5 and 25 ft narrower than in the Type 8-11 Designs.

The Type 13 Design had the same sediment basin as the Type 11 Design. The downstream channel contained vertical retaining walls at the toe of both banks and slightly reduced base widths. The advantage of this design was that the retaining walls would be less than 5-ft high and both the right and left upper banks would remain natural. The base width between the retaining walls was 28 ft between Stations 369+50 and 372+64 widening to 30 ft at Station 373+75 which is the downstream end of the sediment basin. Between Stations 369+50 and 373+75, the Type 13 Design had an excavated invert elevation of 7.0 ft compared to elevation 6.5 in the Type 8-12 Designs.

These plans were evaluated assuming that the 5,400 cfs design discharge would be contained upstream from Lagunitas Road Bridge. This assumption may require the inclusion of floodwalls in the Unit 4 reach.

The Type 11-13 Designs were evaluated using the design annual histograph. The downstream water surface was set at el 2.9 for the 3-day-long design flood and at el 0.5 for the remainder of the year.

The Type 11-13 Designs were evaluated by comparing accumulated deposition in the concrete channel and in the sediment basins at the peak of the design flood and at the end of the one-year simulation. Accumulated deposition for the one-year simulation with the alternative designs is compared to deposition under existing conditions with upstream containment in the following tabulation.

Accumulated Deposition with Design Flood – One-Year Simulation (cu yd)				
	Peak of Design Flood		End of Year	
Plan	Sediment basin	Concrete channel	Sediment basin	Concrete channel
Existing with upstream containment	1,116	58	770	4,651
Type 11	2,628	28	2,710	2,468
Type 12	1,565	28	1,887	2,329
Type 13	2,974	35	2,943	3,762

Water-surface elevations were calculated using the HEC-6W numerical sedimentation model to provide a general idea of how the Type 8-12 Designs would affect water surfaces in the Unit 4 channel. The HEC-6W calculations should be used for comparative purposes only. This model does not account for bridge losses, and the roughness coefficients used in the numerical model were chosen to maximize sediment transport, not to predict water-surface elevations for floods. Calculated water-surface elevations at the peak of the design flood for the Type 8-12 Designs are shown in Figure 17. The water-surface elevation for Type 11 is shown as representative of Types 8-11 for clarity because there is almost no variation in water-surface elevation among the four designs. The top of bank (TOB) was estimated from 2-ft contour interval topographic mapping and should be considered approximate.

The Type 11-13 Designs were less effective than the Type 8-10 Designs in terms of reducing sediment deposition in the concrete channel both at the peak of the design flood and at the end of the annual hydrograph. The advantage of the Type 11-13 Designs was that less disturbance to the natural channel is required for construction.

Type 17, Type 18, and Type 19 Designs

The next three alternatives evaluated using the numerical sediment model were the Type 17, 18 and 19 Designs, which were developed by PWA. The Type 17-19 Designs were the first to incorporate cross sections developed from the 1999 field surveys in the natural channel upstream from the concrete channel. The most significant differences in the new data were the identification of a constriction at Station 375+00 and higher bed elevations in the vicinity of the Lagunitas Road Bridge. The Type 17-19 Designs consisted of deepening and widening the natural channel between the end of the existing concrete channel at Station 369+50 and the beginning of the sediment basin at Station 373+75. The sediment basin in the Type 17 Design was the same as the sediment basin in the Type 11 Design. The Type 18 sediment basin extended further upstream than the Type 17 basin and retained the existing constricted cross sections at Station 375+00 and just upstream from Lagunitas Road Bridge. The Type 19 sediment basin contained the constriction at Station 375+00, but was the same at the Type 17 Design sediment basin upstream from Lagunitas Road Bridge.

Type 17 Design

The Type 17 Design included vertical retaining walls on both sides of the channel between Stations 369+50 and 373+90. This plan was similar to Type 13 Design, with vertical retaining walls on both sides of channel, but the base widths in the Type 17 Design were slightly greater varying between 35 ft at the concrete channel and 40 ft at the sediment basin. The channel was excavated to el 6.5 at Station 370+00 and to el 8.0 at 373+75.

The sediment basin for the Type 17 Design was located between Stations 373+75 and 377+61 and excavated to el 4.5. This is the same sediment basin design as that used for the Type 11 sediment basin. The constriction at Station 375+00 was widened to 40 ft. The constrictions at Stations 376+04 and 376+50 were widened to 35 and 30 ft respectively.

Type 18 Design

The Type 18 Design included a vertical retaining wall on the left bank only between Stations 369+50 and 373+90. The existing right bank was left undisturbed. Channel base widths were slightly less than with the Type 17 Design, varying between 30 and 38 ft. The channel was excavated to the same elevations as for the Type 17 Design.

The sediment basin in Type 18 Design was about 140 ft longer than the Type 17 sediment basin. It extended between Stations 373+75 and 379+00. The sediment basin width was limited by existing banks, which were not disturbed with this plan. The sediment basin contains the constrictions at Station 375+00 (22-ft-wide base width), at Station 376+04 (15-ft-wide base width), and Station 376+50 (20-ft-wide base width). The design allows for a 1V: 1H bank from the existing toe of bank to the sediment basin invert at el 4.5.

Type 19 Design

The Type 19 Design consisted of the Type 18 Design downstream from Lagunitas Road Bridge, and the Type 17 Design upstream from Lagunitas Road Bridge. This retains the existing constriction at Station 375+00, but allows for widening at Stations 376+04 and 376+50.

Type 17-19 Design results

The Type 17-19 Designs were evaluated assuming that the 5,400 cfs design discharge would be contained upstream from Lagunitas Road Bridge. This assumption may require the inclusion of floodwalls in the Unit 4 reach.

The Type 17-19 Designs were evaluated using the design annual histograph. A starting water-surface of el 0.5 was used for most of the annual histograph. A starting water-surface elevation of 2.9 ft was used for the 3-day-long design flood.

Calculated sediment deposition quantities for the Type 17 -19 Designs are compared to calculated deposition under existing conditions with upstream containment in the following tabulation.

Accumulated Deposition with Design Flood – One-Year Simulation (cu yd)				
	Peak of Design Flood		End of Year	
Plan	Sediment basin	Concrete channel	Sediment basin	Concrete channel
Existing with upstream containment	1,116	58	770	4,651
Type 17	2,635	768	2,565	5,588
Type 18	927	314	1,172	3,401
Type 19	1,945	92	1,953	3,616

The numerical model results indicate that the Type 17-19 Designs would convey more sediment into the concrete channel prior to the peak of the design flood than would the existing channel with a floodwall containment structure. This will increase hydraulic roughness at the flood peak. Therefore, in terms of sedimentation effects, these alternatives are less desirable than existing conditions with containment. The advantage of these designs is that water-surface elevations through the natural channel reach upstream from the concrete channel will be lower and floodwalls will therefore not be as high. Constrictions

present in the Type 18 sediment basin were responsible for the low effectiveness of the sediment basin in this plan.

"Minimum Plan"

The "minimum plan" consisted of removing the existing control structure at upstream terminus of the concrete channel and excavating the natural channel about 3 ft upstream through the Lagunitas Road Bridge to Station 377+61. The existing natural bank slope would be retained throughout the excavated reach. A control structure would be constructed across the channel near Station 378+00 to prevent headcutting upstream. The "minimum plan" would contain a flood discharge of 4,100 cfs. Calculations with the numerical model were made assuming a design annual histograph with a peak of 4,100 cfs at the Ross gauge. This histograph had the same shape as the 3-5 January 1982 histograph with all hourly discharges reduced by a factor of 0.5857.

Evaluation of Maintenance Dredging in Concrete Channel

Removal of accumulated sediment in the concrete channel is an expensive endeavor because it requires construction of a coffer dam downstream from the concrete channel, dewatering of about 3,000 ft of channel, pumping of low flows through the dredging reach, and removal of sediment by heavy equipment. It has not been economically feasible for Marin County to remove the sediment on an annual basis.

Significant sediment deposition was calculated in the lower reaches of the concrete channel on the recession limb of the annual design histograph with every alternative studied. The design invert at the downstream end of the concrete channel is 10 ft below mean sea level (msl) and the channel will accumulate sediment when the discharge is less than about 2,500 cfs.

An alternative to annual dredging in the concrete channel is to raise the concrete channel walls in the lower reaches to contain the increased water-surface elevations caused by increased bottom roughness and by loss of conveyance area. The numerical model was used to evaluate the effects of allowing for channel maintenance on 2-, 5-, and 10-year schedules.

Appropriate "average" channel deposition preceding the annual design histograph for each of the maintenance schedules had to be determined. This was accomplished using the 14 available annual histographs from the 1989 sediment study. Four 14-year-long stochastic histographs were developed by sequencing randomly selected annual histographs. The numerical model was used to calculate accumulated sediment deposition in the concrete channel over the 14-year period for each of the four stochastic histographs and the actual historical hydrograph. In each case the Type 17 geometry was used in the model and the sediment basin was dredged at the end of each year. Calculated accumulated

deposition in the concrete channel is shown in Figure 18. It also shows the cumulative average of the five histographs.

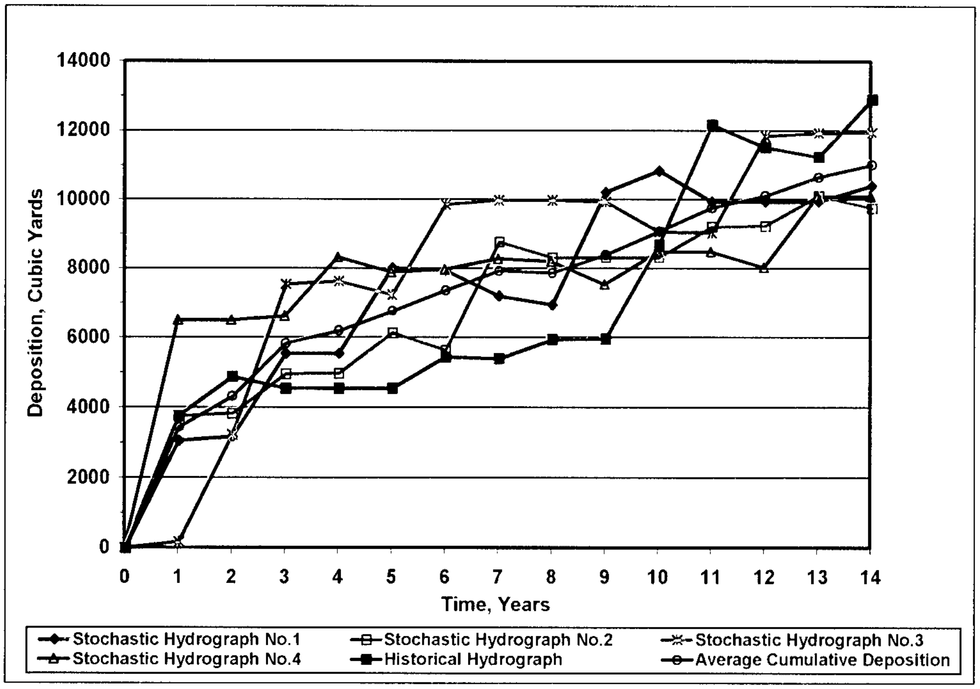

Figure 18. Calculated accumulated deposition in the concrete channel

Average antecedent deposition for each maintenance schedule was calculated in the numerical model using the stochastic histograph with the calculated accumulated deposition closest to the average. It was assumed that the stochastic hydrograph appropriate for the Type 17 geometry would also be appropriate for subsequent design evaluations. To evaluate a 2-Year-maintenance schedule, the first year of stochastic histograph No. 1 was chosen as antecedent flow for the design annual histograph. To evaluate a 5-Year maintenance schedule, the first 4 years of stochastic histograph No. 1 were chosen as antecedent flow for the design annual histograph. To evaluate a 10-Year maintenance schedule, the first 9 years of stochastic histograph No. 2 were chosen as antecedent flow for the design annual histograph.

Average annual dredging in the sediment basin was estimated using the stochastic histographs in the numerical model. Calculated accumulated annual dredging for each of the stochastic hydrographs and the historical hydrograph is shown in Figure 19. The average accumulated dredging is also shown in Figure 19. The total calculated average accumulated dredging over the 14-year simulation was about 19,000 cu yd. Average annual dredging is 1,360 cu yd. Calculated annual dredging in the sediment basin for each of the five histographs is shown in Figure 20. This figure demonstrates the high variability that should be expected in annual dredging quantities in the sediment basin. These calculations were made with the Type 17 Design sediment basin, which was also used in Type 19 Design.

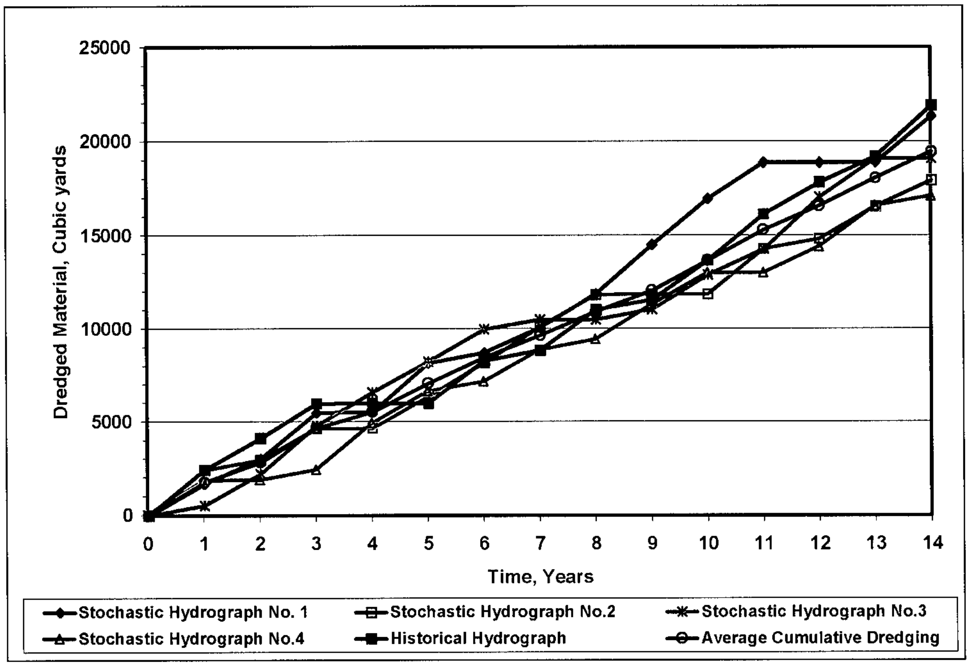

Figure 19. Calculated accumulated deposition in the sediment basin

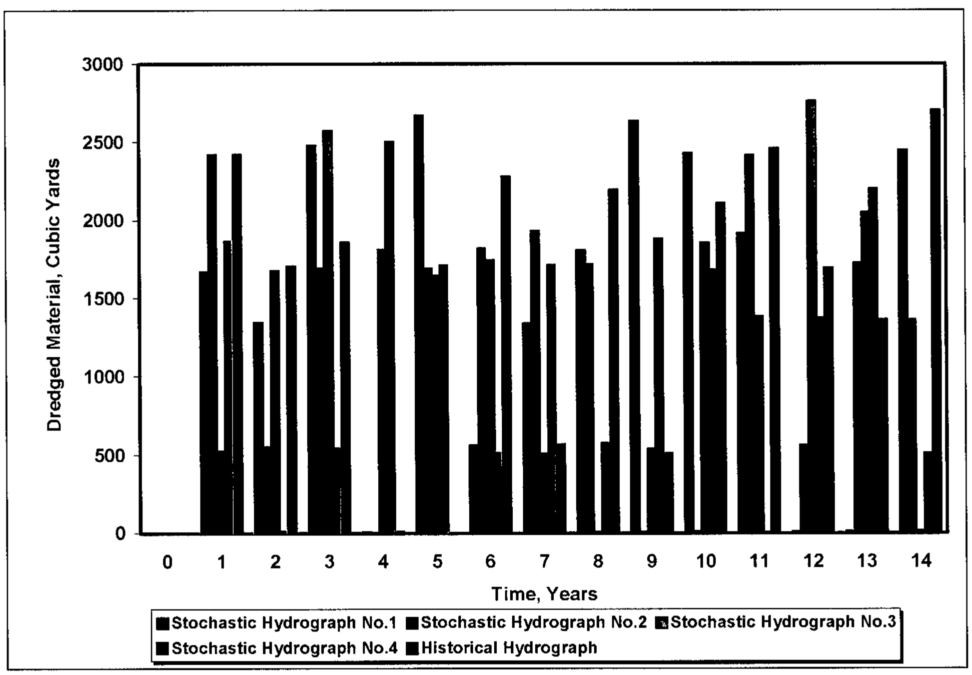

Figure 20. Calculated annual dredging in the sediment basin

Comparison of Existing, "Minimum," and Type 19 Design Plans

Three alternatives were chosen for detailed evaluation with the numerical sediment model. These were the existing condition without containment, minimum plan, and Type 19 Design. Antecedent deposition was calculated for annual, 2-, 5-, and 10-year maintenance schedules in the concrete channel. For each maintenance schedule it was assumed that the sediment basin or excavated channel in the vicinity of Lagunitas Road Bridge would be excavated back to design elevations annually.

The numerical sediment model calculates sediment deposition for every time-step in the histograph at every cross section in the model. Selected locations and times are reported herein. Of particular interest are: (a) antecedent sediment accumulation in the concrete channel with the various maintenance schedules, (b) sediment accumulation at the peak of the design flood, and (c) sediment accumulation at the end of the design annual hydrograph.

Type 19 Design

Antecedent sediment accumulation in the concrete channel for the Type 19 Design and each of the maintenance schedules is shown in Figure 21. This figure demonstrates that the rate of sediment accumulation decreases with time.

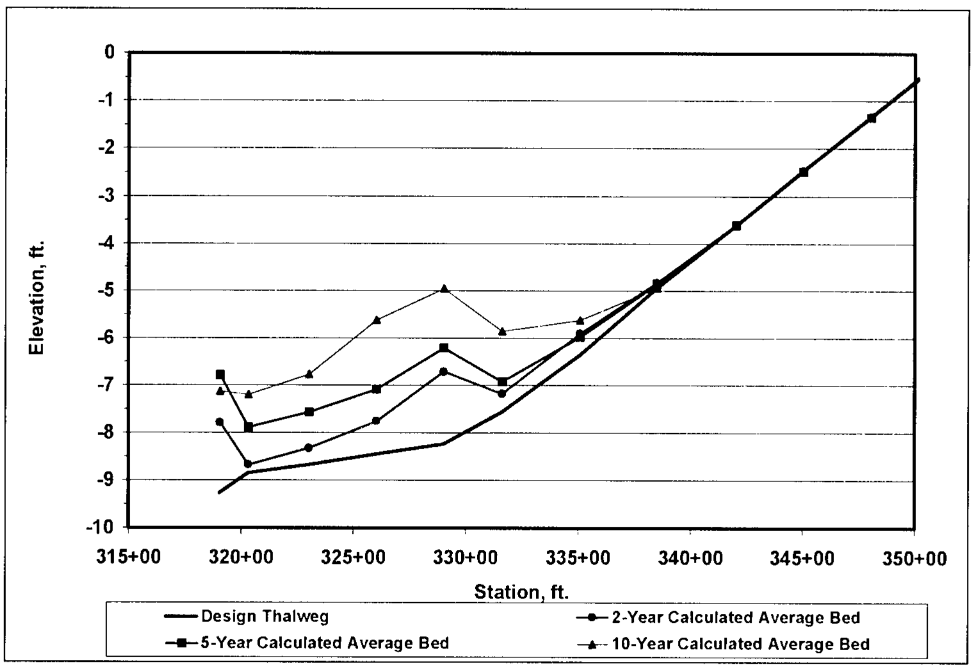

Figure 21. Antecedent aggradation in concrete channel – Type 19 Design

Calculated time histories of the sediment depth at four cross sections during the peak 30 hr of the 3-day-long design flood are shown in Figures 22-25. With annual maintenance and upstream containment, most of the sediment is removed from the concrete channel by the time the design flood peak occurs. However, with continued sediment accumulation, removal of sediment deposits will not occur even at the peak discharge. At Station 319+05, which is located at the downstream end of the concrete channel, sediment removal is almost complete with the annual and 2-year maintenance schedules. However, with the 5-year and 10-year maintenance schedules, no sediment removal is achieved. At Stations 323+00 and 329+00 sediment removal is essentially complete by the time the peak occurs for all but the 10-year maintenance schedule. At Station 335+06 sediment removal is essentially complete by the time the peak occurs for all maintenance schedules.

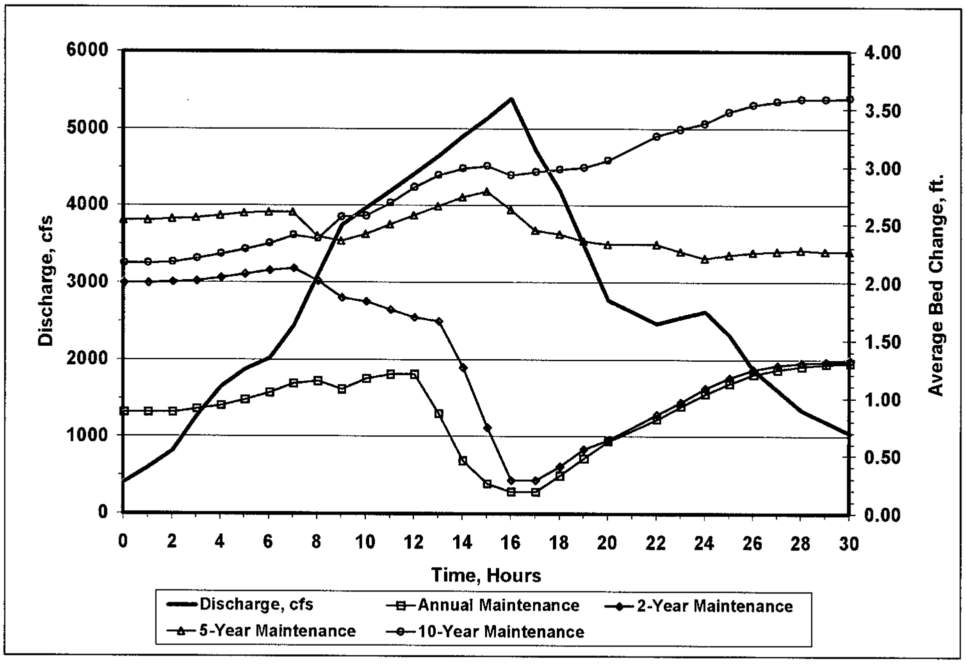

Figure 22. Deposition at Station 319+05 during 5,400-cfs flood – Type 19 Design

The effect of deposited sediment on roughness and conveyance area at the peak of the design flood for the Type 19 Design is reported in Table 1. In the table, deposition depths are rounded to 0.1 ft for incorporation in HEC-RAS geometry to account for conveyance losses. Calculated Manning's roughness coefficients have been adjusted to increase in an upstream direction. Reported roughness coefficients are "expected values," not necessarily design values. Design values should be higher to account for uncertainties in the design water-surface elevation calculations. Calculated water-surface elevations in the concrete channel at the peak of the design flood are shown in Figure 26. These water-surface elevations should be used for comparative purposes only as HEC-6W does not account for bridge losses. The calculations show that concrete-channel wall heights will need to be increased regardless of the maintenance

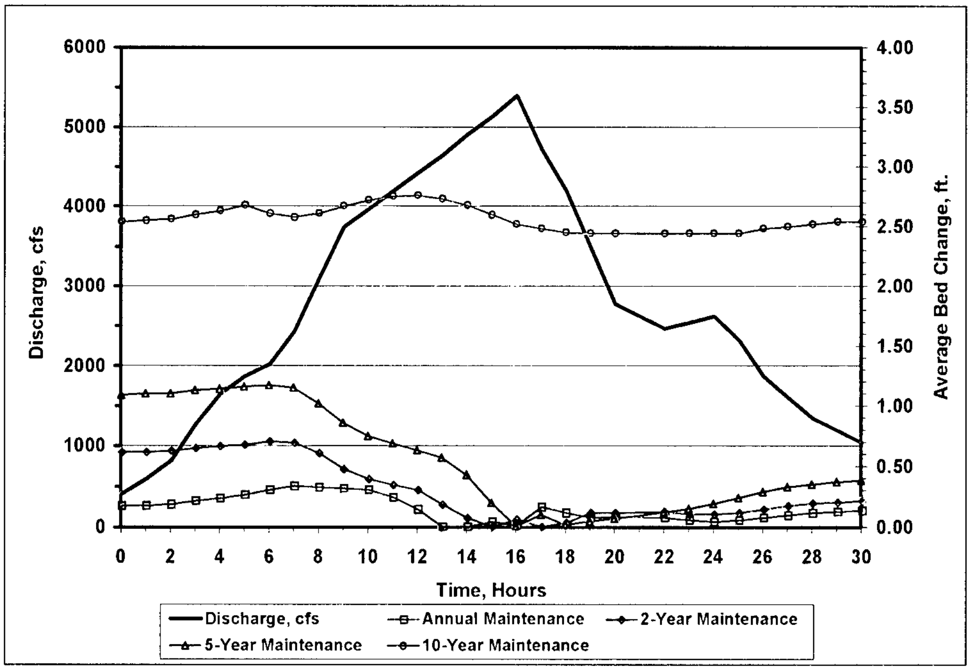

Figure 23. Deposition at Station 323+00 during 5,400-cfs flood – Type 19 Design

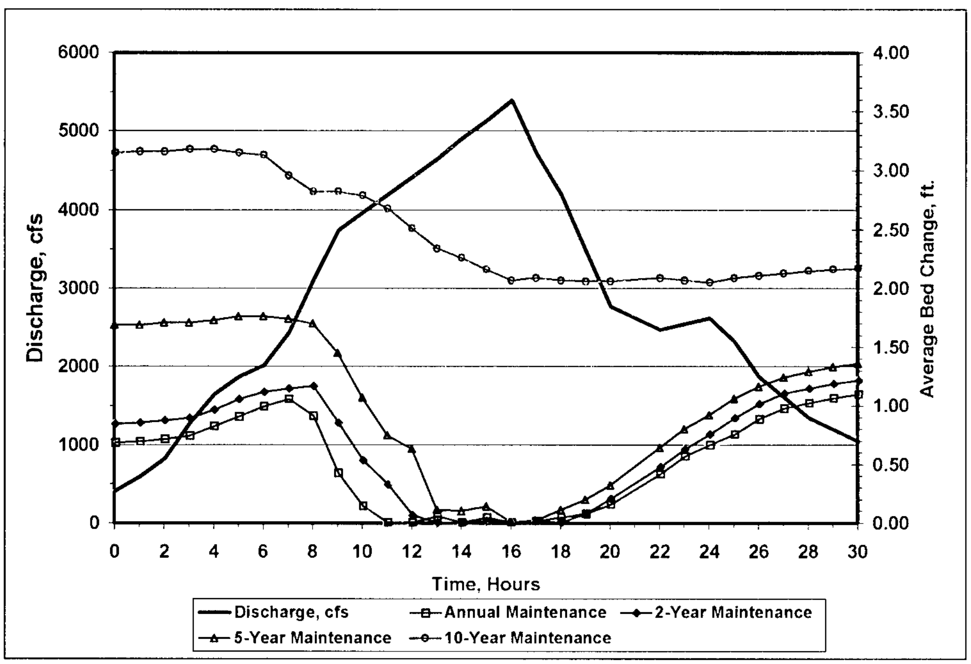

Figure 24. Deposition at Station 329+00 during 5,400-cfs flood – Type 19 Design

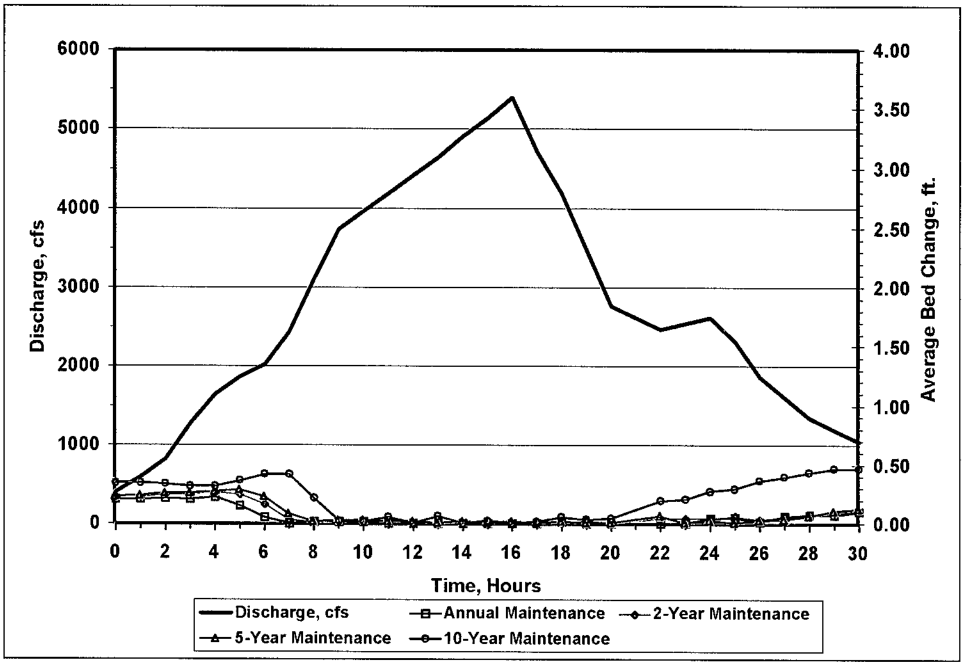

Figure 25. Deposition at Station 335+06 during 5,400-cfs flood – Type 19 Design

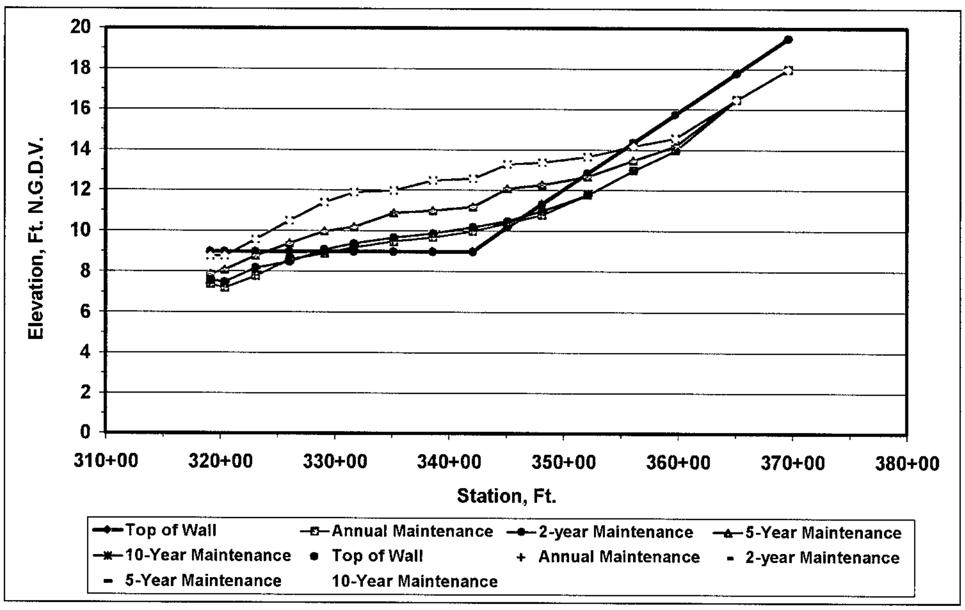

Figure 26. Calculated water-surface elevations at peak of 5,400-cfs flood – Type 19 Design

schedule selected. With a 2-year maintenance schedule, wall heights would have to be about 0.2 ft higher than with annual maintenance. With a 5-year maintenance schedule, wall heights would have to be about 1.3 ft higher than with annual maintenance. With a 10-year maintenance schedule, wall heights would have to be about 2.6 ft higher than with annual maintenance.

Sediment deposition in the concrete channel, in the sediment basin, and in the earthen channel downstream from the concrete channel, at the peak and at the end of the design annual hydrograph are reported in Tables 2-4. Calculated sediment deposition in the earthen channel downstream from the concrete channel includes only sand and gravel deposition. Silts and clays were not simulated in the numerical model. Calculated results in the earthen channel do not represent total sediment deposition and should be used for comparative purposes only. It would be inappropriate to use these numerical model results to estimate the frequency of maintenance dredging in the earthen channel. Computational reaches are associated with the cross sections in the numerical model and represent a reach bounded by the midpoints between the designated cross section and the upstream and downstream cross sections.

"Minimum plan"

Antecedent sediment accumulation in the concrete channel for the "minimum plan" and each of the maintenance schedules is shown in Figure 27. With the "minimum plan" there was an average of 0.5 ft more sediment deposition with the 2-year maintenance schedule and an average of 1.0 ft more sediment deposition with the 5-year and 10-year maintenance schedules than calculated with the Type 19 Design.

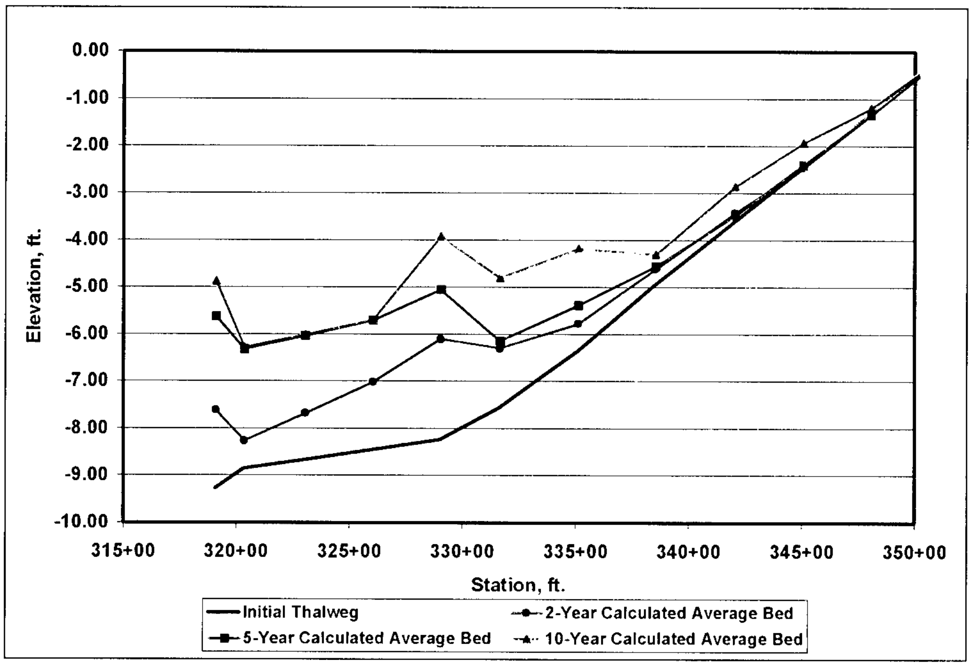

Figure 27. Antecedent aggradation in concrete channel – "minimum plan"

Calculated time histories of the sediment depth at four cross sections during the course of the 30-hr design flood are shown in Figures 28-31. The peak of the design flood with the "minimum plan" is only 4,100 cfs, which is insufficient to remove accumulated sediment, even with annual maintenance, at three of the four cross sections, as shown in the figures. Only at Station 335+06 is sediment

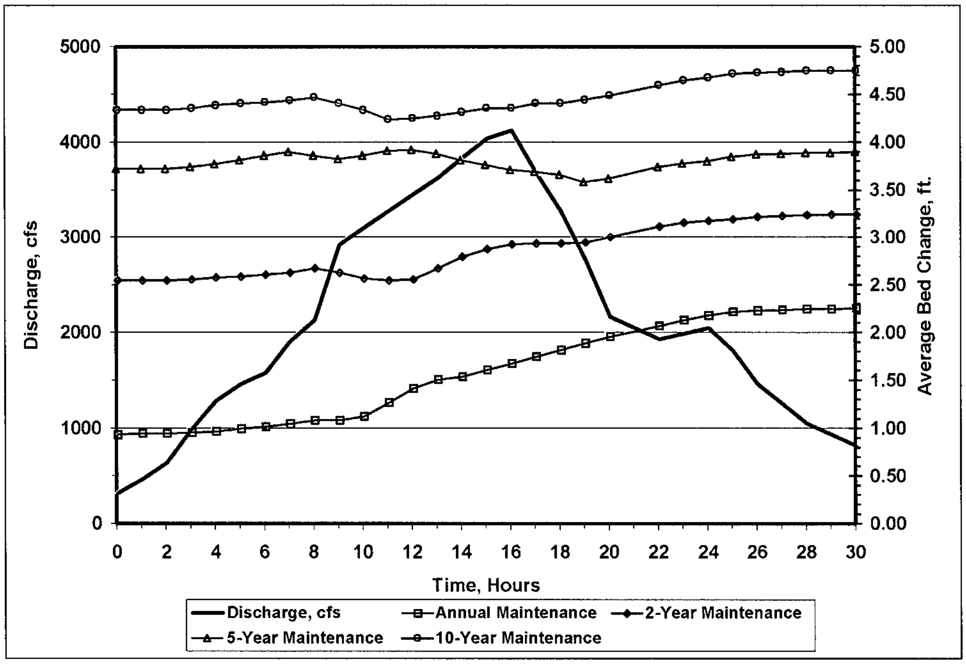

Figure 28. Deposition at Station 319+05 during 4,100-cfs flood – "minimum plan"

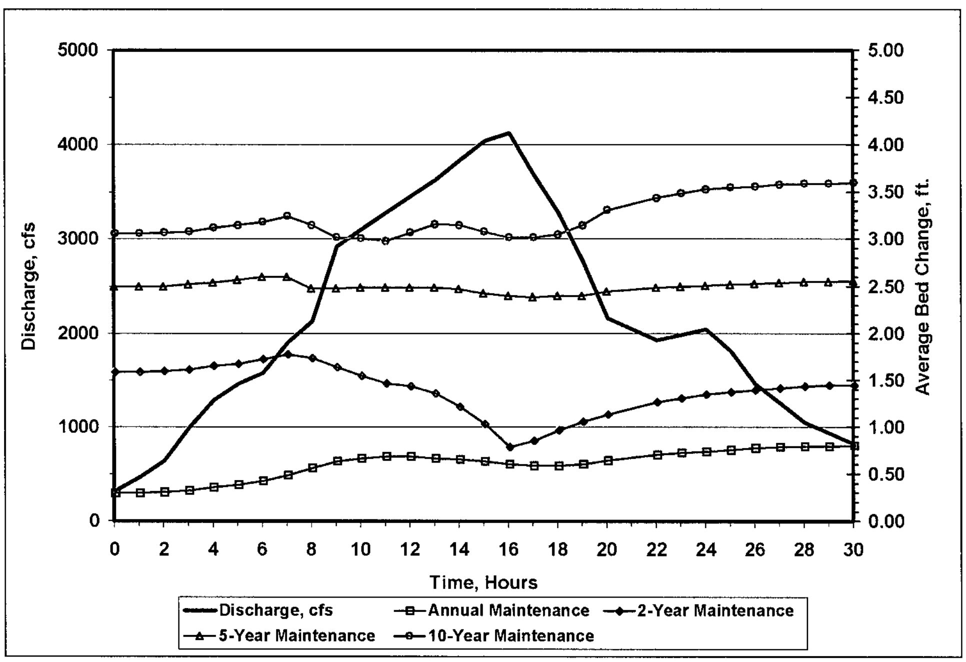

Figure 29. Deposition at Station 323+00 during 4,100-cfs flood – "minimum plan"

removed by the time the peak occurs. With the 10-year maintenance schedule, removal of sediment deposits will not occur even at Station 335+06. The inability of the sediment to wash out of the concrete channel is due to both the lower design discharge and the additional sediment accumulation in the channel caused by the lack of a sediment basin.

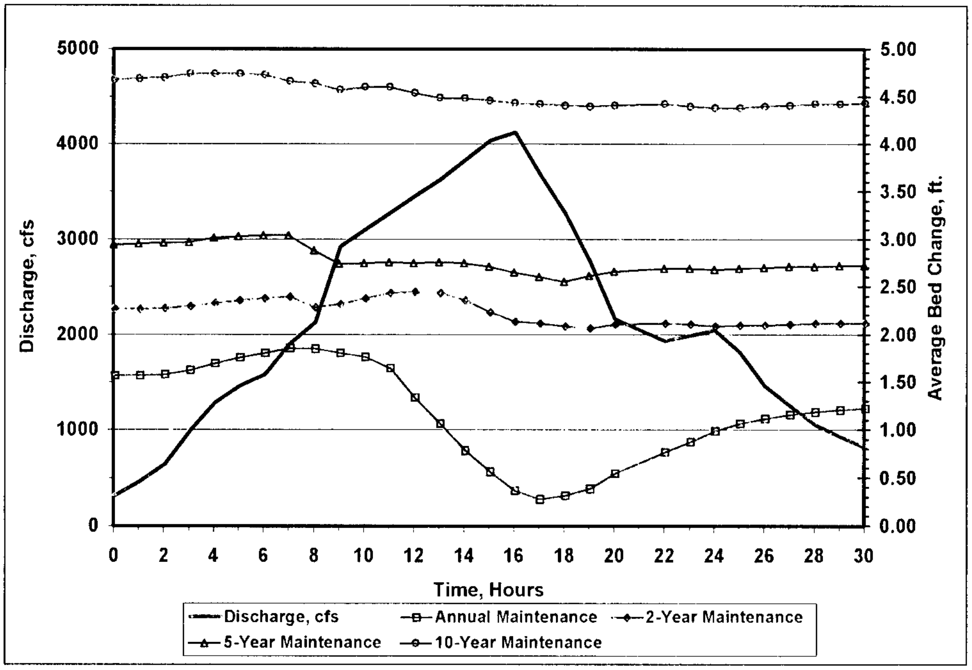

Figure 30. Deposition at Station 329+00 during 4,100-cfs flood - "minimum plan"

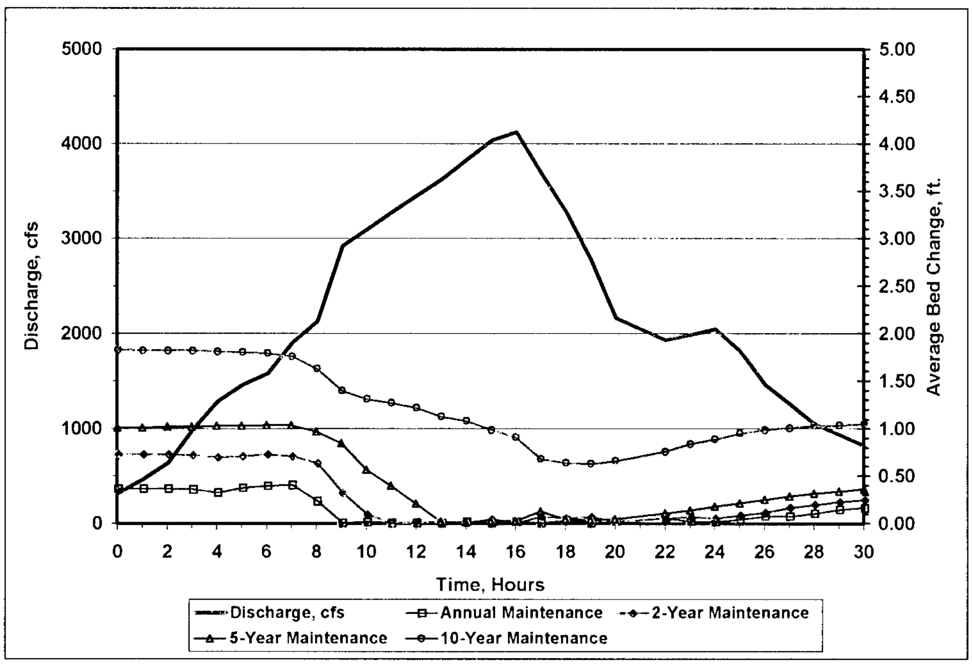

Figure 31. Deposition at Station 335+06 during 4,100-cfs flood - "minimum plan"

The effect of deposited sediment on roughness and conveyance area at the peak of the design flood for the "minimum plan" is reported in Table 5. In the table, deposition depths are rounded to 0.1 ft for incorporation in HEC-RAS geometry to account for conveyance losses. Calculated Manning's roughness coefficients have been adjusted to increase in an upstream direction. Reported roughness coefficients are "expected values" not necessarily design values.

Composite roughness for the "minimum plan" is slightly greater than for the Type 19 Design for the entire length of the concrete channel. It occurs because a greater percentage of the channel perimeter is accounted for by the bed roughness with the lower water depths that accompany the lower discharge of the minimum plan peak flood. The higher roughness coefficients associated with sediment deposition extend about 250 ft further upstream with annual maintenance and about 600 ft further upstream with a 2-year maintenance schedule for the "minimum plan" compared to the Type 19 Design. The upstream extent of the higher roughness coefficients associated with sediment deposition is the same for the "minimum plan" and the Type 19 Design for the 5-year and 10-year maintenance schedules.

Calculated water-surface elevations in the concrete channel at the peak of the design flood (4,100 cfs) are shown in Figure 32. These water-surface elevations should be used for comparative purposes only as HEC-6W does not account for bridge losses. The calculations show that wall heights will need to be increased even with the "minimum plan" regardless of the maintenance schedule selected. With a 2-year maintenance schedule, wall heights would have to be about 0.6 ft higher than with annual maintenance. With a 5-year maintenance schedule, wall heights would have to be about 1.6 ft higher than with annual maintenance. With a 10-year maintenance schedule, wall heights would have to be about 2.7 ft higher than with annual maintenance.

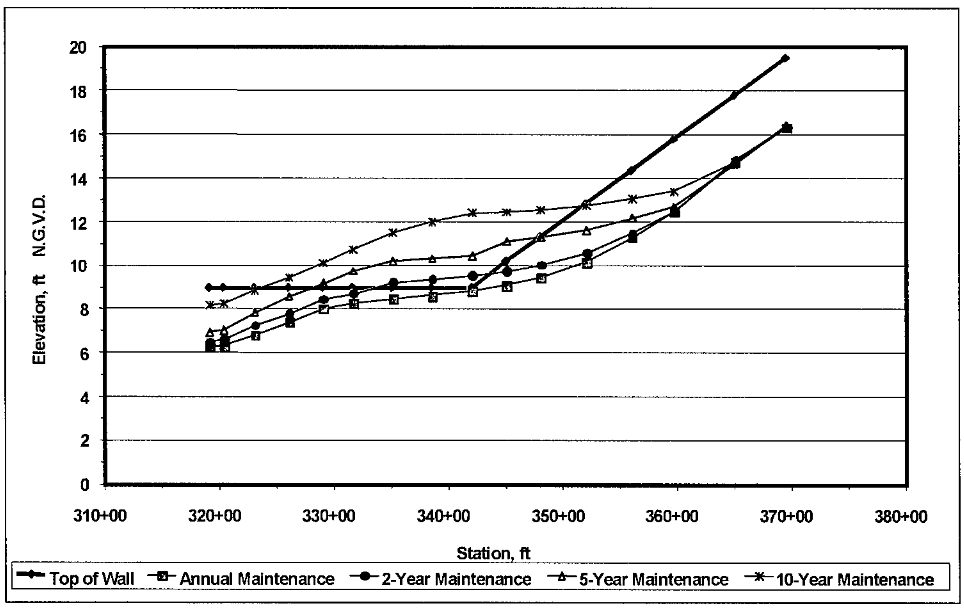

Figure 32. Calculated water-surface elevations at peak of 4,100-cfs flood – "minimum plan"

Sediment deposition at the peak and at the end of the design annual hydrograph for the "minimum plan" is reported in Tables 6-8. Calculated deposition in the concrete channel, in the natural channel upstream from the concrete channel, and in the earthen channel downstream from the concrete channel is reported. Calculated sediment deposition in the earthen channel downstream from the concrete channel includes only sand and gravel deposition. Silts and clays were

not simulated in the numerical model. Calculated results in the earthen channel should be used for comparative purposes only. Computational reaches are associated with the cross sections in the numerical model and represent a reach bounded by the midpoints between the designated cross section and the upstream and downstream cross sections.

Existing conditions

The existing conditions or "do-nothing plan" evaluated in these detailed studies was slightly different from the existing conditions studied initially. The cross section geometry in the natural channel (Unit 4) reach was based on 1988 data from the Sacramento District in the initial numerical model evaluations. The cross section geometry in the natural channel was modified for the detailed evaluations based on field surveys conducted by PWA in 1999. The significant differences in the new geometry were a constricted cross section located at Station 375+50, just downstream from Lagunitas Road Bridge, and higher bed elevations in the vicinity of Lagunitas Road Bridge. The constricted cross section had not been identified in the previous geometric model. The higher bed elevations are in a location where maintenance excavation occurs on an irregular schedule. Geometric differences here are most likely related to timing of the field surveys and the maintenance work.

Current maintenance practice is to excavate the existing channel in the vicinity of Lagunitas Road on an annual basis. In the numerical simulations of existing conditions it was assumed that channel excavation would continue to occur on an annual basis. Cross sections between Stations 373+47 and 376+50 were dredged at the end of each water year in the numerical model. The dredging template was set between el 10 at the downstream cross section and el 12 at the upstream cross section. Initial bed elevations were higher than the specified maintenance elevations at some cross sections, so a dredging operation was simulated in the numerical model before each numerical run. This resulted in 770 cu yd of initial sediment removal.

Antecedent sediment accumulation in the concrete channel for existing conditions and each of the maintenance schedules is shown in Figure 33.

Calculated time histories of the sediment depth at four cross sections during the course of the 30-hr design flood with existing conditions without containment are shown in Figures 34-37. At Stations 319+05, 323+00 and 329+00 sediment deposits remain in the channel at the peak flow for all four maintenance schedules. At Station 335+06 sediment deposits are washed out for the annual, 2-year, and 5-year maintenance schedules, but not for the 10-year maintenance schedule. This lack of sediment removal efficiency is primarily due to breakout upstream at discharges greater than 3,000 cfs.

The effect of deposited sediment on roughness and conveyance area at the peak of the design flood for existing conditions is reported in Table 9. In the table, deposition depths are rounded to 0.1 ft for incorporation in HEC-RAS geometry to account for conveyance losses. Calculated Manning's roughness

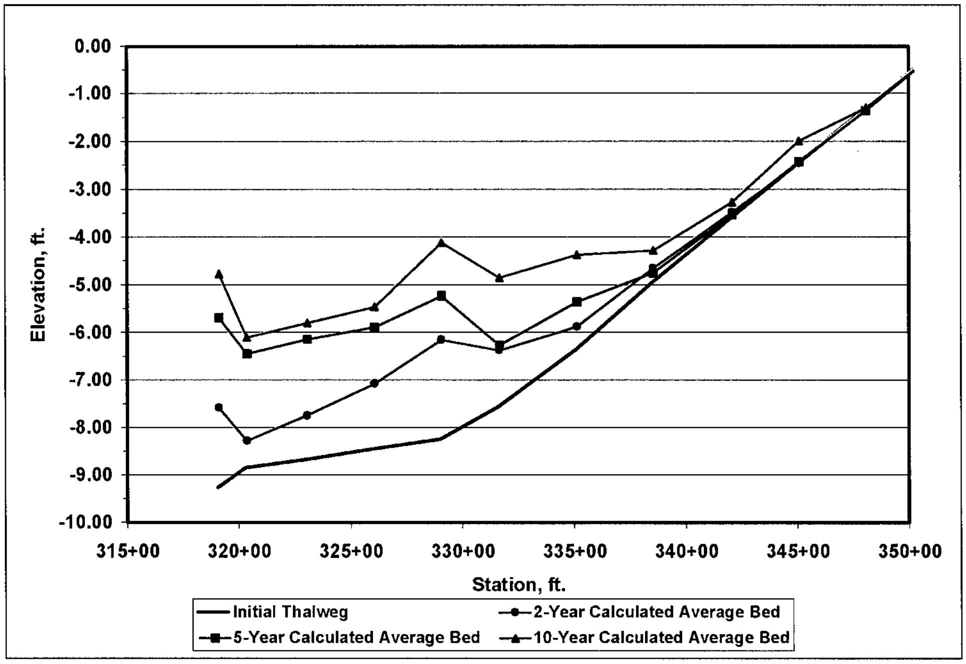

Figure 33. Antecedent aggradation in concrete channel – existing conditions

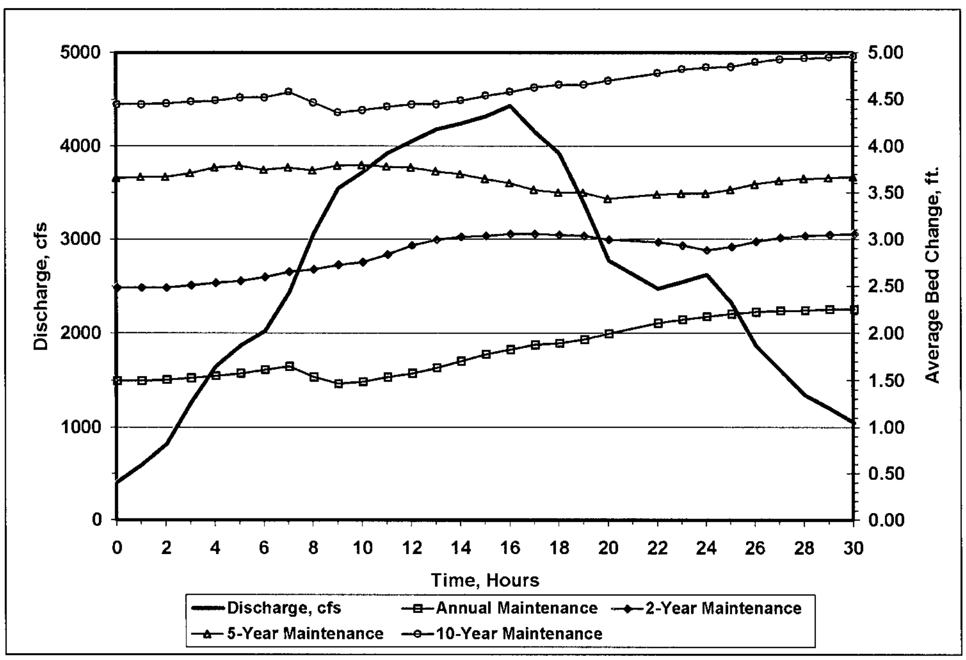

Figure 34. Deposition at Station 319+05 during 5,400-cfs flood – existing conditions

coefficients have been adjusted so that they increase in an upstream direction. Reported roughness coefficients are "expected values" not necessarily design values. Calculated water-surface elevations are lower with existing conditions than for the Type 19 Design because channel discharge is less at the peak flow condition because flow breaks out of the channel upstream.

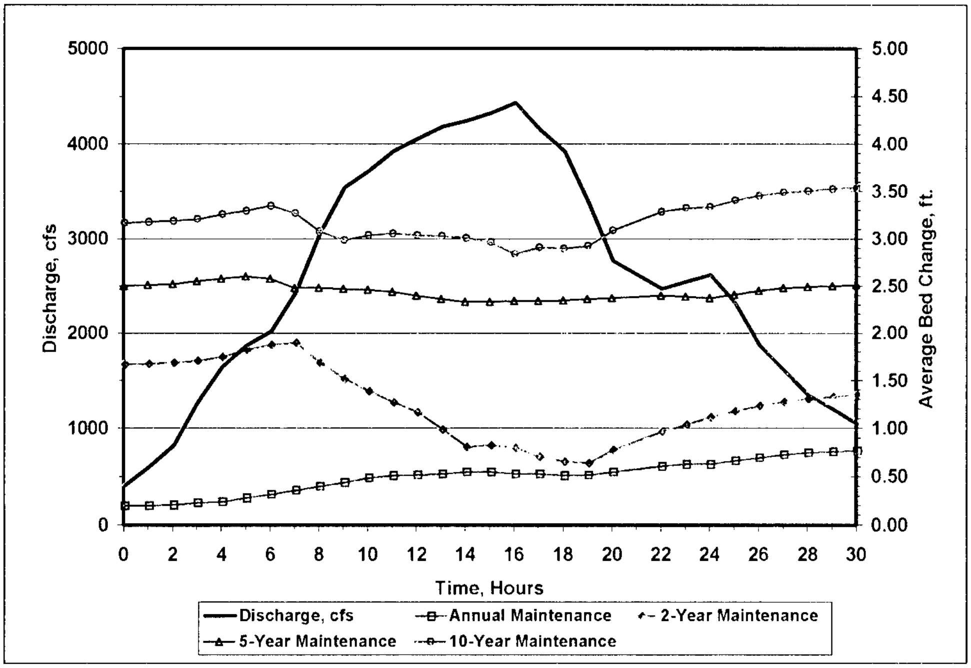

Figure 35. Deposition at Station 323+00 during 5,400-cfs flood – existing conditions

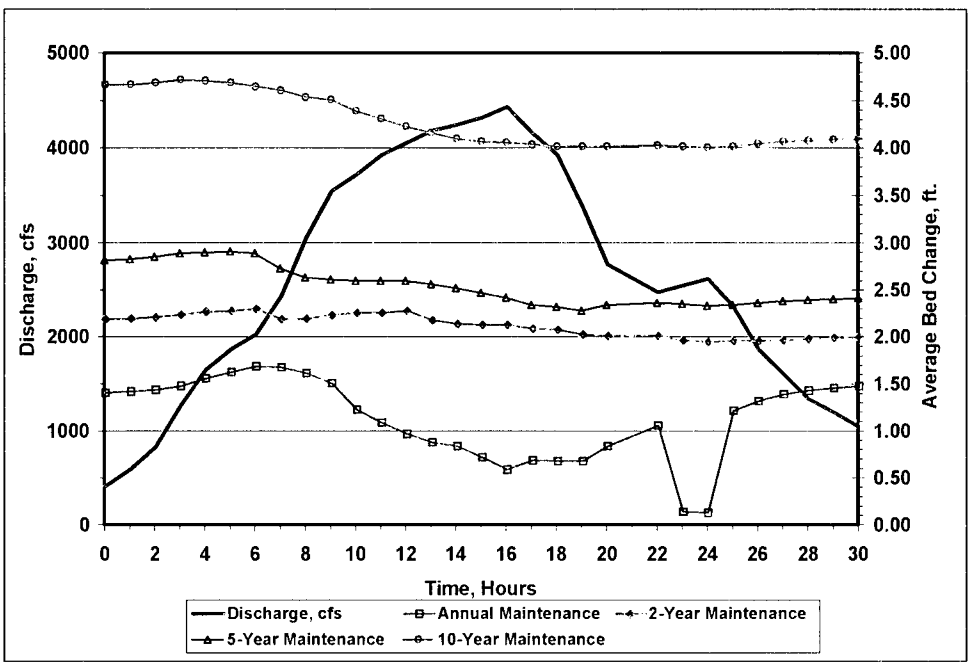

Figure 36. Deposition at Station 329+00 during 5,400-cfs flood – existing conditions

Sediment deposition at the peak and at the end of the design annual hydrograph for existing conditions is reported in Tables 10-12. Calculated deposition in the concrete channel, in the natural sediment basin and natural channel upstream from the concrete channel, and in the earthen channel

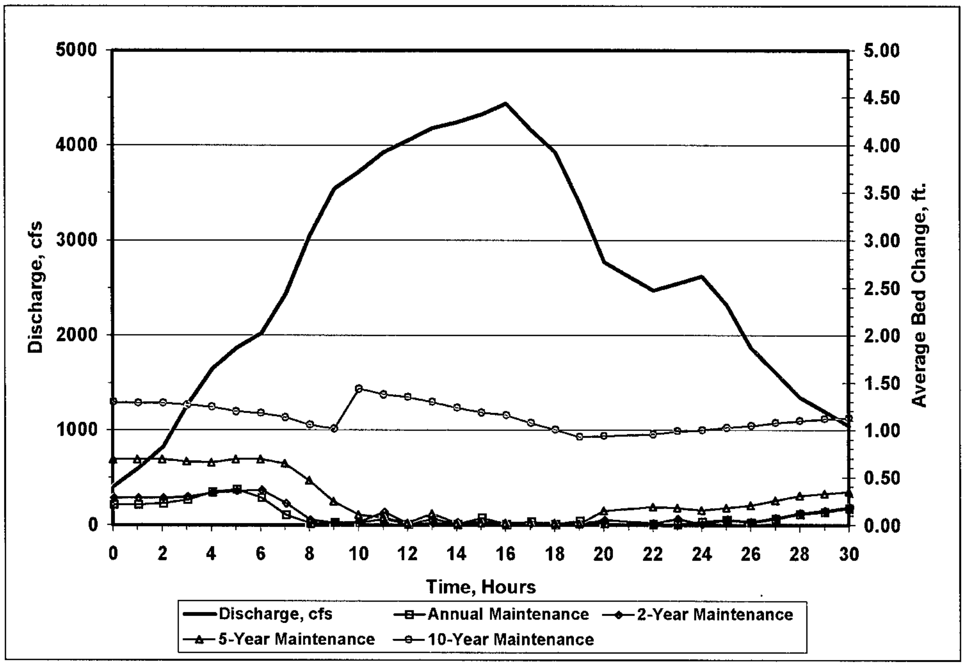

Figure 37. Deposition at Station 335+06 during 5,400-cfs flood – existing conditions

downstream from the concrete channel are reported. Calculated sediment deposition in the earthen channel downstream from the concrete channel includes only sand and gravel deposition. Silts and clays were not simulated in the numerical model. Calculated results in the earthen channel should be used for comparative purposes only. Computational reaches are associated with the cross sections in the numerical model and represent a reach bounded by the midpoints between the designated cross section and the upstream and downstream cross sections.

Annual maintenance in Unit 4

Annual maintenance is expected upstream from the concrete channel in Unit 4 for the Type 19 Design, the "minimum plan" and for the existing conditions. For the Type 19 Design the sediment basins would be excavated back to design elevations every year. For the "minimum plan" the channel between Stations 373+47 and 376+50 would be excavated back to el 8.0 at Station 373+47 increasing to el 9.2 at Station 376+50. It is expected that the current maintenance operations between Stations 373+47 and 376+50 would continue with the "do-nothing," or existing alternative. The existing alternative would excavate the channel to el 10 at Station 373+47 increasing to el 12 at Station 376+50. Annual deposition in the upstream natural channel is dependent on the annual hydrograph. The numerical model calculated annual removal quantities for each year during the simulations for maintenance scenarios. Upstream deposition is independent of deposition in the concrete channel so it is appropriate to average the calculated deposition from the 2-year, 5-year, and 10-year antecedent flow conditions. Calculated average and maximum annual deposition for the three plans are listed in the following tabulation.

Plan	Average Annual Deposition in Unit 4 (cu yd)	Maximum Annual Deposition in Unit 4 (cu yd)
Existing Conditions	183	305
"Minimum plan"	89	244
Type 19 Design	954	1,996

A longer period of record should be evaluated to obtain more reliable annual deposition quantities. The above tabulation should be used for comparative purposes only.

Type 20 Design - Excavation of Bench on Right Bank

The Type 20 Design consists of removing the right wall of the concrete channel down to el 1.0 and excavating a 41-ft-wide bench into the right overbank between Stations 319+00 and 331+00. The bench would have a 24-ft base width and a 1V: 2H side slope. The existing ground is at el 9.5 so the bench would require an average excavation depth of 8.5 ft. There would be a 14 deg expansion on the bench downstream from Station 331+00. The Type 19 Design would be constructed in the Unit 4 reach. Some vegetation would be allowed on the bench to provide an aesthetically pleasing environment. A roughness coefficient of 0.04 was assigned to the bench in the numerical model.

Calculated water-surface elevations at the peak of the 5,400-cfs flood and roughness coefficients for the four maintenance alternatives are tabulated in Table 13. Calculated water-surface elevations in the concrete channel at the peak of the design flood are shown in Figure 38. These water-surface elevations should be used for comparative purposes only as HEC-6W does not account for bridge losses. The calculations show that wall heights will need to be increased regardless of the of the maintenance schedule selected, but not as much as for the Type 19 Design. At Station 342+00, with a 2-year maintenance schedule, wall heights would have to be about 0.1 ft higher than with annual maintenance. At Station 342+00, with a 5-year maintenance schedule, wall heights would have to be about 0.4 ft higher than with annual maintenance. At Station 342+00, with a 10-year maintenance schedule, wall heights would have to be about 1.7 ft higher than with annual maintenance. Calculated water-surface elevations for the Type 20 design are lower than for the Type 19 Design. At Station 342+00 calculated water-surface elevations were 0.4, 0.5, 1.2, and 1.3 ft lower with the Type 20 Design for the annual, 2-year, 5-year, and 10-year maintenance schedules, respectively.

Calculated volumes of sediment deposition in the concrete channel at the peak of the 5,400-cfs flood and at the end of the design hydrograph for the four maintenance alternatives are tabulated in Table 14.

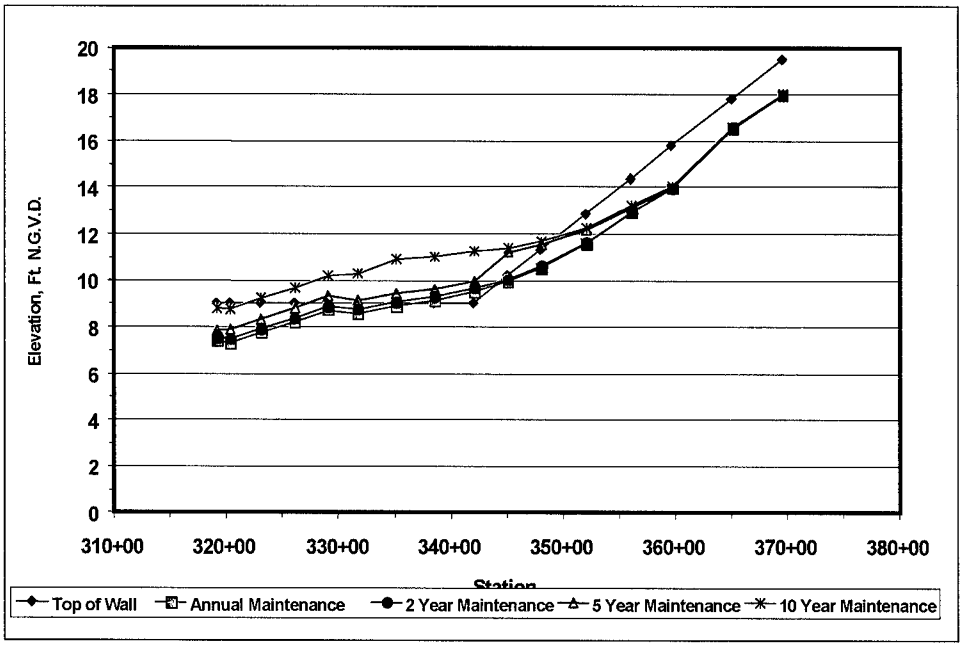

Figure 38. Calculated water-surface elevations at peak of 5,400-cfs flood – Type 20 Design

More sediment is present in the concrete channel at the peak of the design flood with the Type 20 Design than with the Type 19 Design. For example with the annual maintenance alternative, at the peak of the 5,400-cfs flood, the numerical model calculated 670 cu yd of sediment deposition in the concrete channel with the Type 20 Design. This compares to 58 cu yd of calculated deposition with the Type 19 Design. However, calculated roughness coefficients were the same for the Type 19 and Type 20 Designs for all maintenance schedules because sediment deposits occurred in the same reaches. Even a small sediment deposit causes an increase in bottom roughness. The calculated water-surface elevation at the peak of the 5,400-cfs flood at Station 331+60 is about 0.6 ft lower with the Type 20 Design than with the Type 19 Design with annual and 2-year maintenance schedules. It is 1.0 ft lower with a 5-year maintenance schedule and 1.6 ft lower with a 10-year maintenance schedule. Calculated sediment deposition in the concrete channel at the end of the design hydrograph with annual maintenance is 1,036 cu yd more (or 27 percent higher) with the Type 20 Design than with the Type 19 Design.

Summary of Numerical Model Results

The alternatives evaluated in the numerical sedimentation study are listed with an abbreviated description of each plan. The performance of the plans with respect to sedimentation processes can be compared in a summary tabulation of calculated accumulated deposition in the concrete channel and in the sediment basins at the peak of the 5,400-cfs design flood and at the end of the one-year simulation. The calculated quantities in the tabulation all assumed annual maintenance, i.e., a clean channel at the beginning of the numerical simulation.

The plans include the following:

a. Existing -The initial geometry was based on cross sections from HEC-2 backwater models obtained from the Sacramento District in 1988. This geometry was revised in final runs for existing conditions using data from field surveys conducted by PWA in 1999. When the discharge at the Ross gauge exceeds 3,000 cfs flow breaks out of the channel according to a rating curve developed by the Sacramento District in 1988.

b. Existing with containment - The geometry is the same as for existing conditions. All flows are contained in the channel. No breakout occurs upstream from Lagunitas Road Bridge

c. Type 1 - Consists of two sediment basins excavated 3 ft below the existing channel bed. The upstream basin is located between Stations 376+50 and 378+48. The downstream basin is located between Stations 373+47 and 375+00.

d. Type 2 - Consists of two sediment basins excavated 5 ft below the existing channel. The basins are at the same locations as in the Type 1 Design.

e. Type 3 - This is the 1989 Sacramento District "selected plan," that includes channel widening and deepening downstream from Lagunitas Road Bridge and a sediment basin upstream from Lagunitas Road Bridge.

f. Type 4 - Similar to the Type 1 Design, except that the upstream sediment basin is extended to Station 384+09 and excavated 3 ft below existing channel bed.

g. Type 5 - Similar to the Type 2 Design, except that the upstream sediment basin is extended to Station 384+09 and excavated 5 ft below the existing channel bed.

h. Type 6 - The channel downstream from Lagunitas Road Bridge is excavated to el 6.5 retaining natural banks. The Type 2 upstream sediment basin is retained.

i. Type 7 - The channel downstream from Lagunitas Road Bridge is from the Sacramento District's "selected plan." The Type 2 upstream sediment basin is retained.

j. Type 8 - The channel between Stations 369+50 and 377+61 is widened between 30 and 60 ft and excavated to el 6.5. There is no sediment basin.

k. Type 9 - The channel between Stations 369+50 and 373+75 is widened between 30 and 35 ft and excavated to el 6.5. The sediment basin is located between Stations 373+75 and 377+61, is 45- to 60-ft-wide, and excavated to el 4.5.

l. Type 10 - Same as Type 9, except that the sediment basin is excavated to el 1.5.

m. Type 11 - Same as Type 9, except that the sediment basin width upstream from Lagunitas Bridge is reduced from 60 ft to 30 ft. (75-ft-wide at Station 375+88 transitions to 30-ft-wide at Station 376+50 and then remains 30-ft-wide to Station 377+61.)

n. Type 12 - Same as Type 11, except that the channel width between Stations 369+50 and 373+75 is reduced from 30-35 ft to 25-30 ft.

o. Type 13 - Same as Type 11, except that the channel width between Stations 369+50 and 373+75 is reduced from 30-35 ft to 28-30 ft and the channel is excavated to el 7.0 instead of el 6.5.

p. Type 17 - PWA Plan, with channel widening and excavation between concrete channel and sediment basin. Similar to Type 13 Design, with vertical retaining walls on both sides of channel. Base widths vary between 35 and 40 ft, which is wider than Type 13 Design. Channel excavated to el 6.5 at Station 370+00 and to el 8.0 at 373+75. Sediment basin between Stations 373+75 and 377+61 excavated to el 4.5 (same as Type 11 sediment basin). Constriction at Station 375+00 widened to 40 ft.

q. Type 18 - PWA Plan, with channel widening and excavation between concrete channel and sediment basin, using retaining wall on left bank only. Base width slightly less than with Type 17, varying between 30 and 38 ft. Channel and sediment basin excavated to same elevations as Type 17 Design. Sediment basin lengthened from Type 17, extending between Stations 373+75 and 379+00. Sediment basin width limited by existing banks. Basin contains the constriction at Station 375+00 (22-ft-wide base width), and at Station 376+04 (15-ft-wide base width).

r. Type 19 - Consists of the Type 18 Design downstream from Lagunitas Road Bridge, and the Type 11 Design upstream from Lagunitas Road Bridge.

s. Type 20 - Consists of the Type 19 Design in Unit 4 and a bench excavated into the right overbank at the downstream end of the concrete channel.

t. "Minimum Plan" - The channel between Stations 369+50 and 377+61 is excavated about 3 ft retaining the natural bank slope. The design flow is 4,100 cfs.

The following tabulation is a summary of the accumulated deposition for the design flood.

Accumulated Deposition with the 5,400-cfs Design Flood - One Year Simulation (cu yd)				
	Peak of Design Flood		End of Year	
Plan	**Sediment Basin**	**Concrete Channel**	**Sediment Basin**	**Concrete Channel**
Existing	1,071	879	625	4,466
Upstream Containment	1,116	58	770	4,651
Type 1	1,731	32	1,321	4,835
Type2	1,908	29	1,901	4,563
Type 3	5,480	13	5,367	4,572
Type 4	1,875	28	1,906	4,519
Type 5	2,067	22	3,162	3,602
Type 6	1,764	24	830	3,995
Type 7	11	40	134	2,702
Type 8	2,644	21	2,101	2,952
Type 9	4,053	14	3,843	2,577
Type 10	5,515	9	6,400	2,005
Type 11	2,628	28	2,710	2,468
Type 12	1,565	28	1,887	2,329
Type 13	2,974	35	2,943	3,762
Type 17	2,635	768	2,569	5,591
Type 18	927	314	1,172	3,401
Type 19	1,945	92	1,953	3,616
Type 20	1,945	750	1,953	4,872
"Minimum Plan"	712	1,190	141	4,374

4 Conclusions

Sediment deposition occurs in the lower reaches of Corte Madera Creek because the elevation of the channel bottom is below sea level. These sediment deposits reduce the flood-carrying capacity of the channel by reducing the conveyance area of the channel and by causing an increase in hydraulic roughness on the channel bottom. Additional roughness is added to the channel side walls below sea level by the presence of tube worms and barnacles. The significance of the increase in roughness is related to the quantity and composition of the deposited material.

The numerical sedimentation model, HEC-6W, can be used to evaluate the significance of maintenance plans and channel modifications on project performance.

Annual removal of sediment deposits and aquatic growth from the concrete channel will reduce hydraulic roughness in the concrete channel during the 5,400-cfs design annual hydrograph. With annual maintenance, most of the sediment deposited in the concrete channel from antecedent flow can be washed out of the channel by the time the flood peak occurs. The most important feature of the flood control project, in terms of allowing for sediment deposits to be washed out, is upstream containment of breakout flows. Under existing conditions, when the discharge exceeds about 3,000 cfs, flow breaks out of Corte Madera Creek and flows away from the channel. Containment of breakout flows will require construction of floodwalls and/or channel improvement in the Unit 4 reach. Without upstream containment, the deposited sediment will not wash out of the concrete channel for the design flood even with annual cleanout. Another important feature of the flood control project, in terms of allowing sediment deposits to be washed out, is a sediment basin upstream from the concrete channel. To be effective, the sediment basin must have sufficient capacity to trap most of the coarsest sediment on the rising limb of the annual hydrograph. Coarse sediment deposits are more difficult to re-entrain and move out of the concrete channel than fine sediment deposits.

Most of the sediment basins evaluated in this study were small and did not trap sufficient quantities of sediment to allow for complete removal of the sediment deposits in the concrete channel. The sediment basin originally designed by the Sacramento District (Type 3) and the Type 10 sediment basin performed the best, allowing for almost complete removal of concrete channel sediment deposits by the time the peak of the 5,400-cfs design annual flood arrived.

Maintenance costs for this project will be high and careful attention to obtaining a reliable operation and maintenance plan is recommended. A more detailed study is required to obtain average annual maintenance quantities. A longer period of record should be used to develop the stochastic hydrographs.

Degradation was not allowed to occur in the natural channel upstream from the end of the concrete channel in the HEC-6W numerical model. Previous studies have indicated that the final channel design will need to include invert armoring and toe protection for modified banklines.

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Table 1
Type 19 Design
Calculated Water-Surface Elevation, Bed Change, and Roughness Coefficient at Peak of 5,400-cfs Flood

Station ft	Annual Maintenance WSEL ft	Annual Maintenance Deposition ft	Annual Maintenance n-value	2-Year Maintenance WSEL NGVD	2-Year Maintenance Deposition ft	2-Year Maintenance n-value	5-Year Maintenance WSEL ft	5-Year Maintenance Deposition ft	5-Year Maintenance n-value	10-Year Maintenance WSEL ft	10-Year Maintenance Deposition ft	10-Year Maintenance n-value
319+05	7.4	0.2	0.029	7.7	0.6	0.029	8.0	2.8	0.030	8.8	3.0	0.030
320+30	7.3	0	0.029	7.6	0	0.029	8.2	0	0.030	8.8	2.3	0.030
323+00	8.0	0	0.029	8.2	0.1	0.029	8.8	0	0.029	9.6	2.6	0.030
326+00	8.6	0.1	0.029	8.6	0	0.029	9.4	0.2	0.029	10.6	2.5	0.030
329+00	9.1	0	0.029	9.2	0	0.029	10.0	0	0.029	11.4	2.1	0.030
331+60	9.2	0	0.019	9.4	0	0.019	10.2	0	0.028	11.9	0	0.030
335+06	9.5	0	0.019	9.7	0	0.019	10.9	0	0.028	12.4	0.1	0.030
338+48	9.7	0	0.019	9.9	0	0.019	11.0	0	0.028	12.5	0	0.030
342+00	10.0	0	0.019	10.2	0	0.019	11.2	0	0.027	13.1	0	0.030
345+00	10.4	0	0.018	10.5	0	0.018	12.1	0	0.027	13.3	0	0.030
348+00	10.8	0	0.018	11.0	0	0.018	12.3	0	0.018	13.4	0	0.018
352+00	11.8	0	0.018	11.8	0	0.018	12.7	0	0.018	13.7	0	0.018
356+00	13.0	0	0.018	13.0	0	0.018	13.5	0	0.018	14.2	0	0.018
359+66	14.1	0	0.018	14.1	0	0.018	14.2	0	0.018	14.6	0	0.018
365+00	16.5	0	0.018	16.5	0	0.018	16.5	0	0.018	16.4	0	0.018
369+50	18.0	0	0.018	18.0	0	0.018	18.0	0	0.018	18.0	0	0.018

Table 2
Type 19 Design
Calculated Deposition in Concrete Channel at Peak and End of 5,400-cfs Flood

Annual Maintenance			2-Year Maintenance			5-Year Maintenance			10-Year Maintenance		
Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd
369+50	0	0	369+50	0	0	369+50	2	0	369+50	0	1
365+00	0	0	365+00	0	0	365+00	4	0	365+00	0	1
359+66	0	0	359+66	0	0	359+66	4	1	359+66	1	0
356+00	0	0	356+00	0	0	356+00	4	0	356+00	1	1
352+00	0	0	352+00	0	0	352+00	4	1	352+00	2	0
348+00	0	0	348+00	0	0	348+00	4	1	348+00	3	1
345+00	0	1	345+00	1	2	345+00	4	2	345+00	4	8
342+00	4	8	342+00	4	41	342+00	7	60	342+00	8	106
338+48	4	31	338+48	4	132	338+48	4	192	338+48	4	488
335+06	5	267	335+06	5	375	335+06	6	509	335+06	12	774
331+60	4	511	331+60	4	567	331+60	5	639	331+60	7	764
329+00	17	988	329+00	5	990	329+00	47	1,051	329+00	745	1,137
326+00	4	728	326+00	22	739	326+00	3	771	326+00	929	898
323+00	18	451	323+00	4	461	323+00	70	502	323+00	910	925
320+30	5	193	320+30	5	211	320+30	27	245	320+30	565	620
319+05	31	438	319+05	168	451	319+05	801	570	319+05	864	1,168
Total	92	3,616	Total	222	3,969	Total	996	4,544	Total	4,055	6,272

Table 3
Type 19 Design
Calculated Deposition in Sediment Basin at Peak and End of 5,400-cfs Flood

Annual Maintenance			2-Year Maintenance			5-Year Maintenance			10-Year Maintenance		
Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd
377+50	1	0	377+50	2	0	377+50	3	1	377+50	36	35
377+25	234	294	377+25	528	582	377+25	1,078	1,126	377+25	2,213	2,266
376+50	307	376	376+50	711	774	376+50	1,372	1,428	376+50	2,669	2,736
375+88	663	510	375+88	964	840	375+88	2,000	1,869	375+88	2,656	2,524
375+50	574	383	375+50	912	608	375+50	1,462	1,165	375+50	2,504	2,273
375+00	3	102	375+00	12	94	375+00	98	190	375+00	158	318
374+27	161	250	374+27	386	505	374+27	640	661	374+27	1,541	1,575
373+90	2	38	373+90	72	102	373+90	93	150	373+90	137	173
Total	1,945	1,953	Total	3,587	3,505	Total	6,746	6,590	Total	11,914	11,900

Table 4
Type 19 Design
Calculated Deposition in Earthen Channel Downstream from Concrete Channel at Peak and End of 5,400-cfs Flood

Annual Maintenance			2-Year Maintenance			5-Year Maintenance			10-Year Maintenance		
Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd
317+10	5,246	5,576	317+10	7,191	7,269	317+10	8,487	8,483	317+10	12,873	12,821
310+00	6,652	9,907	310+00	8,396	10,421	310+00	11,697	12,959	310+00	18,299	18,744
303+00	353	2,540	303+00	1,020	3,531	303+00	2,938	5,981	303+00	9,077	10,979
293+00	4,809	8,263	293+00	7,459	11,180	293+00	10,751	14,616	293+00	23,930	28,308
280+00	4,640	6,899	280+00	5,835	8,668	280+00	10,024	13,007	280+00	18,555	21,838
260+00	1,294	1,659	260+00	1,486	1,908	260+00	2,653	3,326	260+00	12,993	15,376
238+00	506	756	238+00	639	887	238+00	1,008	1,234	238+00	6,480	6,860
222+00	527	698	222+00	617	799	222+00	841	1,045	222+00	6,586	6,834
201+00	-69	-77	201+00	-64	-72	201+00	-55	-60	201+00	-430	-438
190+00	-92	-107	190+00	-93	-107	190+00	-98	-111	190+00	-486	-494
181+00	-22	-28	181+00	-23	-28	181+00	-26	-30	181+00	-454	-447
166+40	256	310	166+40	261	315	166+40	283	336	166+40	2,037	2,079
Total	24,100	36,396	Total	32,724	44,771	Total	48,503	60,786	Total	109,460	122,460

Table 5
"Minimum Plan"
Calculated Water-Surface Elevation, Bed Change, and Roughness Coefficient at Peak of Minimum Flood (4,100 cfs)

Station ft	Annual Maintenance WSEL ft	Annual Maintenance Deposition ft	Annual Maintenance n value	2-Year Maintenance WSEL ft	2-Year Maintenance Deposition ft	2-Year Maintenance n value	5-Year Maintenance WSEL ft	5-Year Maintenance Deposition ft	5-Year Maintenance n value	10-Year Maintenance WSEL ft	10-Year Maintenance Deposition ft	10-Year Maintenance n value
319+05	6.3	1.7	0.030	6.5	2.9	0.031	7.0	3.7	0.032	8.2	4.4	0.031
320+30	6.3	0.3	0.030	6.7	0.9	0.031	7.1	2.4	0.032	8.3	2.9	0.031
323+00	6.8	0.6	0.030	7.3	0.8	0.031	7.8	2.4	0.031	8.9	3.0	0.031
326+00	7.4	0.7	0.030	7.8	1.9	0.031	8.6	2.3	0.031	9.5	3.9	0.031
329+00	8.0	0.4	0.030	8.5	2.1	0.031	9.2	2.7	0.030	10.1	4.4	0.031
331+60	8.3	0.0	0.029	8.7	0.1	0.029	9.7	0.3	0.030	10.7	2.6	0.031
335+06	8.5	0.0	0.019	9.2	0.0	0.029	10.2	0.0	0.029	11.5	0.9	0.031
338+48	8.6	0.0	0.019	9.4	0.0	0.019	10.3	0.1	0.029	12.0	0.0	0.031
342+00	8.9	0.0	0.019	9.6	0.0	0.019	10.5	0.0	0.029	12.4	0.0	0.029
345+00	9.1	0.0	0.019	9.8	0.0	0.019	11.1	0.0	0.029	12.5	0.1	0.029
348+00	9.5	0.0	0.019	10.0	0.0	0.019	11.3	0.0	0.019	12.6	0.0	0.019
352+00	10.2	0.0	0.019	10.6	0.0	0.019	11.6	0.0	0.019	12.8	0.0	0.019
356+00	11.3	0.0	0.019	11.5	0.0	0.019	12.2	0.0	0.019	13.1	0.0	0.019
359+66	12.5	0.0	0.019	12.5	0.0	0.019	12.7	0.0	0.019	13.4	0.0	0.019
365+00	14.8	0.0	0.019	14.9	0.0	0.019	14.7	0.0	0.019	14.8	0.0	0.019
369+50	16.4	0.0	0.019	16.3	0.0	0.019	16.4	0.0	0.019	16.3	0.0	0.019

Table 6
"Minimum Plan"
Calculated Deposition in Concrete Channel at Peak and End of Minimum Flood

Annual Maintenance Station ft	Annual Maintenance Peak cu yd	Annual Maintenance End cu yd	2-Year Maintenance Station ft	2-Year Maintenance Peak cu yd	2-Year Maintenance End cu yd	5-Year Maintenance Station ft	5-Year Maintenance Peak cu yd	5-Year Maintenance End cu yd	10-Year Maintenance Station ft	10-Year Maintenance Peak cu yd	10-Year Maintenance End cu yd
369+50	0	0	369+50	0	0	369+50	0	0	369+50	0	0
365+00	0	0	365+00	0	0	365+00	2	0	365+00	1	0
359+66	0	1	359+66	0	0	359+66	0	0	359+66	2	2
356+00	0	0	356+00	0	0	356+00	0	0	356+00	4	1
352+00	0	3	352+00	1	1	352+00	2	0	352+00	3	1
348+00	0	5	348+00	3	3	348+00	4	5	348+00	6	53
345+00	4	10	345+00	4	30	345+00	4	4	345+00	10	192
342+00	3	174	342+00	4	88	342+00	6	168	342+00	4	439
338+48	4	170	338+48	10	235	338+48	20	379	338+48	11	735
335+06	10	352	335+06	5	497	335+06	13	667	335+06	383	1,013
331+60	4	538	331+60	47	633	331+60	120	754	331+60	967	917
329+00	126	1,040	329+00	737	1,067	329+00	914	1,088	329+00	1,527	1,571
326+00	256	708	326+00	692	918	326+00	863	1,080	326+00	1,424	1,582
323+00	214	415	323+00	277	642	323+00	839	974	323+00	1,058	1,403
320+30	70	220	320+30	220	397	320+30	596	657	320+30	706	951
319+05	499	738	319+05	865	957	319+05	1,047	1,161	319+05	1,292	1,481
Total	1,190	4,374	Total	2,865	5,468	Total	4,430	6,937	Total	7,398	10,341

Table 7
"Minimum Plan"
Calculated Deposition in Natural Channel Upstream from Concrete Channel at Peak and End of Minimum Flood

Annual Maintenance			2-Year Maintenance			5-Year Maintenance			10-Year Maintenance		
Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd
377+61	1	0	377+61	2	0	377+61	2	0	377+61	1	0
376+50	2	1	376+50	2	1	376+50	2	1	376+50	20	19
375+88	245	38	375+88	267	85	375+88	291	86	375+88	493	316
375+50	454	47	375+50	446	21	375+50	570	144	375+50	642	255
375+00	3	9	375+00	3	6	375+00	103	133	375+00	260	291
374+27	1	0	374+27	1	0	374+27	5	3	374+27	3	2
373+47	1	1	373+47	2	2	373+47	7	13	373+47	88	87
372+64	2	1	372+64	2	6	372+64	3	5	372+64	0	1
372+12	1	27	372+12	2	75	372+12	1	84	372+12	0	55
371+12	1	16	371+12	1	2	371+12	2	1	371+12	0	3
370+50	0	1	370+50	1	2	370+50	2	2	370+50	2	1
370+00	1	0	370+00	1	1	370+00	1	1	370+00	3	2
Total	712	141	Total	730	201	Total	989	473	Total	1,512	1,032

Table 8
"Minimum Plan"
Calculated Deposition in Earthen Channel Downstream from Concrete Channel at Peak and End of Minimum Flood

Annual Maintenance			2-Year Maintenance			5-Year Maintenance			10–Year Maintenance		
Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd
317+10	4,736	5,369	317+10	6,575	6,790	317+10	9,406	9,382	317+10	14,698	14,533
310+00	5,631	8,268	310+00	7,685	10,233	310+00	11,056	12,941	310+00	18,909	18,906
303+00	337	1,825	303+00	986	3,124	303+00	3,056	5,193	303+00	10,045	11,369
293+00	5,053	8,364	293+00	7,088	10,024	293+00	10,979	14,442	293+00	24,838	28,762
280+00	1,949	3,382	280+00	3,562	5,759	280+00	8,251	10,788	280+00	18,363	21,376
260+00	604	793	260+00	815	990	260+00	1,251	1,528	260+00	10,300	11,994
238+00	284	419	238+00	425	573	238+00	833	998	238+00	6,247	6,478
222+00	145	175	222+00	187	222	222+00	320	364	222+00	5,928	5,982
201+00	-22	-25	201+00	-23	-25	201+00	-25	-27	201+00	-406	-409
190+00	-30	-33	190+00	-32	-36	190+00	-42	-45	190+00	-462	-463
181+00	-1	-1	181+00	-2	-2	181+00	-4	-4	181+00	-469	-469
166+40	71	79	166+40	77	85	166+40	101	110	166+40	1,901	1,908
Total	18,757	28,615	Total	27,343	37,737	Total	45,182	55,670	Total	109,892	119,967

Table 9
Existing Conditions
Calculated Water-Surface Elevation, Bed Change, and Roughness Coefficient at Peak of 5,400-cfs Flood

	Annual Maintenance			2-Year Maintenance			5-Year Maintenance			10-Year Maintenance		
Station ft	WSEL ft	Deposition ft	n value	WSEL ft	Deposition ft	n value	WSEL ft	Deposition ft	n value	WSEL ft	Deposition ft	n value
319+05	7.3	1.8	0.030	7.4	3.1	0.030	7.8	3.6	0.030	8.9	4.6	0.031
320+30	7.3	0.1	0.030	7.5	0.8	0.030	7.8	2.3	0.030	9.0	2.9	0.031
323+00	7.8	0.5	0.029	8.1	0.8	0.030	8.6	2.3	0.030	9.6	2.9	0.031
326+00	8.3	0.7	0.029	8.5	2.1	0.030	9.3	2.3	0.030	10.2	3.9	0.031
329+00	8.8	0.6	0.029	9.3	2.1	0.030	9.9	2.4	0.030	10.9	4.1	0.031
331+60	8.9	0.2	0.029	9.8	0.0	0.029	10.4	0.4	0.029	11.5	2.2	0.030
335+06	9.5	0.0	0.029	10.3	0.0	0.029	10.8	0.0	0.029	12.1	1.1	0.030
338+48	9.9	0.0	0.026	10.8	0.0	0.029	11.2	0.1	0.029	12.6	0.1	0.030
342+00	10.1	0.0	0.019	10.9	0.0	0.029	11.8	0.0	0.029	13.0	0.1	0.030
345+00	10.3	0.0	0.019	11.6	0.0	0.019	12.1	0.0	0.029	13.4	0.0	0.030
348+00	10.6	0.0	0.019	11.8	0.0	0.019	12.8	0.0	0.019	13.8	0.0	0.030
352+00	11.1	0.0	0.019	12.1	0.0	0.019	13.0	0.0	0.019	13.9	0.0	0.019
356+00	12.0	0.0	0.019	12.7	0.0	0.019	14.2	0.0	0.019	14.2	0.0	0.019
359+66	12.8	0.0	0.019	13.2	0.0	0.019	14.4	0.0	0.019	14.4	0.0	0.019
365+00	15.2	0.0	0.019	15.1	0.0	0.019	15.4	0.0	0.019	15.5	0.0	0.019
369+50	16.8	0.0	0.019	16.8	0.0	0.019	19.1	0.0	0.019	16.8	0.0	0.019

Table 10
Existing Conditions
Calculated Deposition in Concrete Channel at Peak and End of 5,400-cfs Flood

Annual Maintenance			2-Year Maintenance			5-Year Maintenance			10-Year Maintenance		
Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd
369+50	0	0	369+50	0	0	369+50	5	3	369+50	0	0
365+00	0	0	365+00	0	0	365+00	6	2	365+00	1	0
359+66	0	0	359+66	0	0	359+66	7	2	359+66	2	0
356+00	0	0	356+00	1	0	356+00	6	4	356+00	4	1
352+00	2	0	352+00	3	0	352+00	6	1	352+00	7	5
348+00	4	1	348+00	4	2	348+00	8	3	348+00	7	46
345+00	5	3	345+00	7	16	345+00	5	66	345+00	7	284
342+00	6	190	342+00	4	114	342+00	21	154	342+00	21	432
338+48	13	180	338+48	7	260	338+48	6	361	338+48	53	701
335+06	6	368	335+06	8	475	335+06	17	622	335+06	482	971
331+60	68	533	331+60	10	606	331+60	124	696	331+60	813	785
329+00	202	967	329+00	728	1,019	329+00	826	1,018	329+00	1,396	1,448
326+00	259	699	326+00	790	819	326+00	852	995	326+00	1,418	1,517
323+00	186	442	323+00	266	711	323+00	819	940	323+00	1,026	1,394
320+30	35	236	320+30	189	422	320+30	573	655	320+30	715	967
319+05	541	703	319+05	903	912	319+05	1,004	1,127	319+05	1,371	1,569
Total	1,327	4,322	Total	2,920	5,356	Total	4,285	6,649	Total	7,323	10,120

Table 11
Existing Conditions
Calculated Deposition in Natural Channel Upstream from Concrete Channel at Peak and End of 5,400-cfs Flood

Annual Maintenance			2-Year Maintenance			5-Year Maintenance			10-Year Maintenance		
Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd
377+61	2	1	377+61	2	1	377+61	3	0	377+61	3	1
376+50	2	1	376+50	2	0	376+50	3	1	376+50	4	1
375+88	3	69	375+88	93	164	375+88	104	173	375+88	415	490
375+50	367	75	375+50	471	233	375+50	770	512	375+50	907	680
375+00	2	0	375+00	2	1	375+00	3	1	375+00	11	9
374+27	4	1	374+27	29	36	374+27	175	173	374+27	493	497
373+47	1	0	373+47	3	1	373+47	5	3	373+47	98	95
372+64	0	0	372+64	0	0	372+64	2	1	372+64	2	1
372+12	0	1	372+12	1	2	372+12	5	2	372+12	4	1
371+12	0	0	371+12	0	0	371+12	1	1	371+12	0	0
370+50	0	0	370+50	0	0	370+50	1	1	370+50	0	0
370+00	0	0	370+00	0	0	370+00	3	1	370+00	0	0
Total	381	148	Total	603	438	Total	1,075	869	Total	1,937	1,775

Table 12
Existing Conditions
Calculated Deposition in Earthen Channel Downstream from Concrete Channel at Peak and End of 5,400-cfs Flood

Annual Maintenance			2-Year Maintenance			5-Year Maintenance			10-Year Maintenance		
Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd
317+10	6,624	6,469	317+10	8,081	8,250	317+10	10,632	10,002	317+10	15,930	14,681
310+00	5,546	10,056	310+00	7,614	10,802	310+00	11,133	13,670	310+00	19,188	22,555
303+00	83	2,377	303+00	781	3,637	303+00	2,814	6,451	303+00	9,753	12,456
293+00	4,497	7,914	293+00	7,248	10,924	293+00	10,782	14,765	293+00	24,354	28,901
280+00	4,512	6,580	280+00	5,528	8,130	280+00	9,985	12,887	280+00	18,173	19,968
260+00	1,310	1,660	260+00	1,541	2,030	260+00	2,652	3,315	260+00	11,645	13,462
238+00	506	756	238+00	639	888	238+00	1,008	1,234	238+00	6,547	6,741
222+00	527	698	222+00	617	800	222+00	841	1,045	222+00	6,579	6,805
201+00	-69	-77	201+00	-64	-72	201+00	-55	-60	201+00	-430	-438
190+00	-92	-107	190+00	-93	-107	190+00	-98	-111	190+00	-486	-494
181+00	-22	-28	181+00	-23	-28	181+00	-26	-30	181+00	-454	-447
166+40	256	310	166+40	261	315	166+40	283	336	166+40	2,038	2,078
Total	23,678	36,608	Total	32,130	45,569	Total	49,951	63,504	Total	112,837	126,268

Table 13
Type 20 Design
Calculated Water-Surface Elevation, Bed Change, and Roughness Coefficient at Peak of 5,400-cfs Flood

Station ft	Annual Maintenance WSEL ft	Annual Maintenance Deposition ft	Annual Maintenance n value	2-Year Maintenance WSEL ft	2-Year Maintenance Deposition ft	2-Year Maintenance n value	5-Year Maintenance WSEL ft	5-Year Maintenance Deposition ft	5-Year Maintenance n value	10-Year Maintenance WSEL ft	10-Year Maintenance Deposition ft	10-Year Maintenance n value
319+05	7.5	0.3	0.029	7.6	0.9	0.029	7.9	2.4	0.030	8.8	3.1	0.030
320+30	7.4	0	0.029	7.6	0.2	0.029	7.9	0.9	0.030	8.8	2.6	0.030
323+00	7.8	0.3	0.029	8.0	0.7	0.029	8.4	1.3	0.030	9.3	2.9	0.030
326+00	8.3	0.8	0.029	8.4	1.2	0.029	8.9	1.9	0.030	9.8	3.8	0.030
329+00	8.8	0.7	0.029	9.0	1.1	0.029	9.4	2	0.030	10.3	4.3	0.030
331+60	8.6	0	0.019	8.8	0	0.019	9.3	0	0.029	10.4	0	0.030
335+06	8.9	0	0.019	9.1	0	0.019	9.6	0	0.028	10.9	0	0.030
338+48	9.2	0	0.018	9.3	0	0.018	9.8	0	0.027	11.0	0	0.030
342+00	9.6	0	0.018	9.7	0	0.018	10.1	0	0.027	11.3	0	0.030
345+00	10.0	0	0.018	10.1	0	0.018	11.4	0	0.027	11.4	0	0.018
348+00	10.5	0	0.018	10.6	0	0.018	11.7	0	0.018	11.7	0	0.018
352+00	11.6	0	0.018	11.7	0	0.018	12.3	0	0.018	12.3	0	0.018
356+00	13.0	0	0.018	13.0	0	0.018	13.2	0	0.018	13.2	0	0.018
359+66	14.1	0	0.018	14.1	0	0.018	14.1	0	0.018	14.1	0	0.018
365+00	16.5	0	0.018	16.6	0	0.018	16.7	0	0.018	16.6	0	0.018
369+50	18.0	0	0.018	18.0	0	0.018	18.0	0	0.018	18.1	0	0.018

Table 14
Type 20 Design
Calculated Deposition in Concrete Channel at Peak and End of 5,400-cfs Flood

Annual Maintenance			2-Year Maintenance			5-Year Maintenance			10-Year Maintenance		
Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd	Station ft	Peak cu yd	End cu yd
369+50	0	0	369+50	0	0	369+50	2	0	369+50	0	0
365+00	0	0	365+00	0	0	365+00	4	0	365+00	0	0
359+66	0	0	359+66	0	0	359+66	4	1	359+66	0	0
356+00	0	0	356+00	0	0	356+00	4	0	356+00	0	0
352+00	0	0	352+00	0	0	352+00	4	1	352+00	0	1
348+00	0	0	348+00	0	1	348+00	4	1	348+00	1	1
345+00	2	0	345+00	5	1	345+00	7	2	345+00	4	66
342+00	4	34	342+00	5	11	342+00	4	56	342+00	4	176
338+48	4	84	338+48	4	86	338+48	4	162	338+48	5	462
335+06	5	360	335+06	5	351	335+06	5	495	335+06	5	744
331+60	4	552	331+60	4	587	331+60	9	686	331+60	7	868
329+00	247	1,173	329+00	418	1,188	329+00	697	1,252	329+00	1,521	1,604
326+00	298	989	326+00	452	1,119	326+00	705	1,150	326+00	1,418	1,710
323+00	114	654	323+00	260	775	323+00	456	1,026	323+00	1,061	1,520
320+30	6	381	320+30	53	416	320+30	220	574	320+30	648	928
319+05	66	645	319+05	213	648	319+05	659	810	319+05	808	1,167
Total	750	4,872	Total	1,419	5,183	Total	2,788	6,216	Total	5,482	9,247

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13. SUPPLEMENTARY NOTES

14. ABSTRACT

Sediment deposition occurs in the lower reaches of the concrete-lined Corte Madera Creek flood control channel because the elevation of the channel bottom is below sea level. These sediment deposits, combined with the presence of tube worms and barnacles on the channel walls, reduce the flood-carrying capacity of the channel. The upstream portion of the original flood control project was not completed and flood flows above 3,000 cfs are not contained in the natural channel upstream. Thus, there is reduced flow competency to carry the coarse sediment load delivered by flood flows. With annual maintenance and upstream containment of breakout flows, most of the sediment deposited in the concrete channel from seasonal antecedent flow can be washed out by the time the flood peak occurs. An HEC-6 numerical sedimentation model study was conducted to evaluate several alternative plans to provide flood containment and sediment storage while also maintaining the natural characteristics of the upstream channel.

15. SUBJECT TERMS

Corte Madera	HEC-6	Sedimentation
Flood control	Numerical model	Sediment basin

16. SECURITY CLASSIFICATION OF:			**17. LIMITATION OF ABSTRACT**	**18. NUMBER OF PAGES**	**19a. NAME OF RESPONSIBLE PERSON**
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